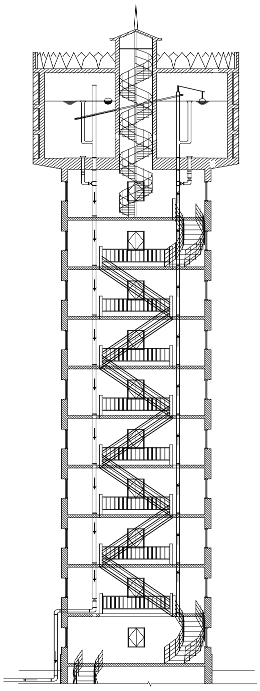
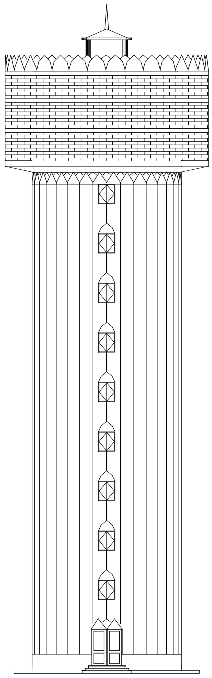
Reinforced Concrete IV
Second Semester
Structural Engineering Department
Faculty of Engineering
Tanta University

General Notes and Solved Examples





By

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First Edition

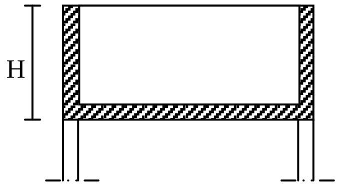
1. Fundamentals

1. Fundamentals

(1)

(a) Statical system for each tank

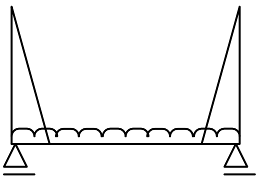
Fig. 1



For H = 1.5 m

The walls of the tank can be treated as cantilever , i.e., the total load acting in each wall will be transmitted in vertical direction

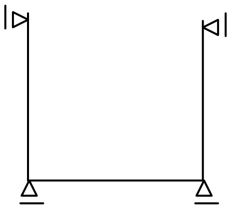
Statical system in conjunction with the loads



Vertical Sec.

For H = 4 m

Top horizontal beam should be used. In this case, the wall will be considered as two way slab.



Vertical strip

Horizontal strip at the sec. of max. horizontal load

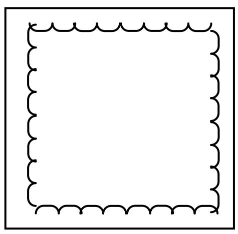
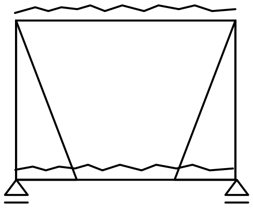


Fig. 2



Statical system for
vl strip (Box. Sec)

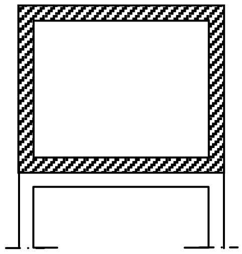
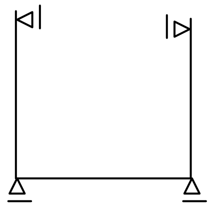
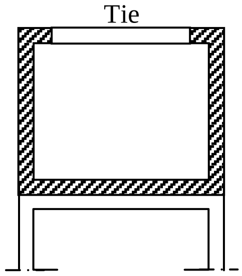


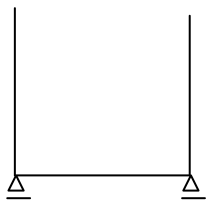
Fig. 3



Structural system
for vl. strip

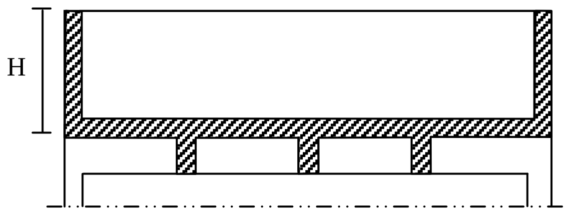


* If the tie is removed



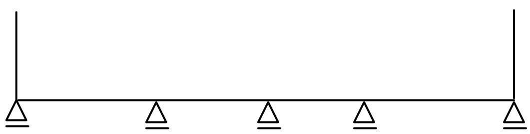
Statical system for vertical strip

Fig. 4



H=2m

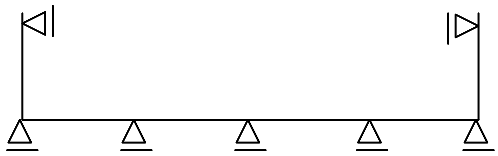
The walls of the tank can be considered as cantilever walls



Statical system for vl. Strip

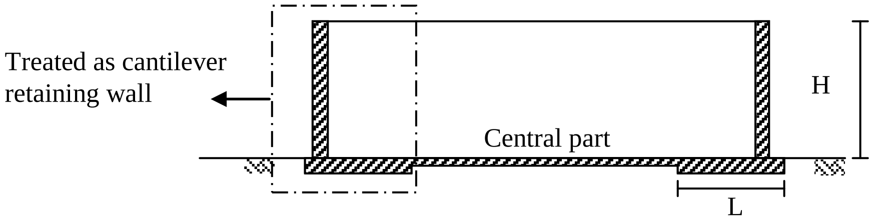
* H = 4 m

Top horizontal beam should be used.



vl. Strip

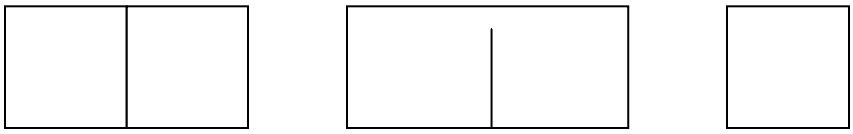
Fig. 5



For large diameter tank, the part of floor with the length (L) with the wall adjacent to it can be assumed as a retaining wall connecting to the central part.

* If the tank has sliding base, the wall and floor will be treated separately.

Fig. 6



Horizontal Strip vl. Strip in long dir. vl. Strip in short dir.

b. Acting loads & case of loading

- Fig. 1 to Fig. 4 are elevated tanks

Acting loads are: * own weight of concrete elements
* water load → vl. Load on floor
→ lateral load on walls
* External lateral load → wind pressure
→ Earthquake Excitation

- case of loading

1- Case of water load (No external lateral load)
2- Empty tank and consider lateral load.

- Fig. 5 is rest on soil tank

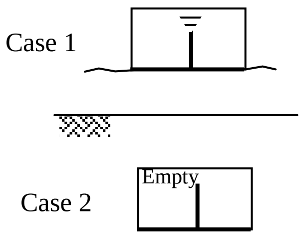
  Acting loads are : own weight of tank
  : Water load
  : Soil reaction

  there is only one case of loading under the effect of all acting loads

- Fig. 6 is under ground tank

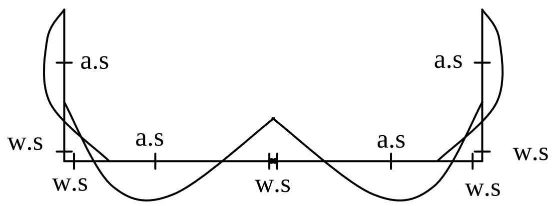
  Acting load are :
  - Own weight of concrete
  - Weight of water
  - Weight of soil above the tank
  - Lateral earth pressure
  - Lateral hydrostatic pressure due to ground water.
  - Soil reaction below the tank
  - Uplift force due to ground water

Case of loading



- tank is full of water and no soil around tank (Check bearing capacity of soil)
- tank is empty and consider soil around tank (Check uplift)

(2)



- The sections that labeled by (w.s) are water side sections.
- The sections that labeled by (a.s) are air side sections.

(3)

sec. 1 $M_u = 0.0$, $N_u = +150$ kN } Water side section

$$T = \frac{T_u}{1.5} = \frac{15}{1.5} = 100 \text{ kN}$$

$$\text{- t} = 0.6\,\text{T} = 0.6*10 = 6\,\text{cm} < 20\,\text{cm}$$

$$\text{take} \quad t = 20\,\text{cm}$$

$$\text{- use } \Phi 10 \rightarrow \beta_{cr} = 0.85$$

$$A_s = \frac{T_u}{\beta_{cr} f_y / \gamma_s} = \frac{150*1.15*1000}{0.85*360} = 563.7 \text{ mm}^2 = 5.637 \text{ cm}^2$$

$$A_s\text{/side} = 5.64/2 = 2.82 \text{ cm}^2$$

$$\text{use } 5\Phi 10 \text{ /m`/ side}$$

- Check of tensile stress for concrete.

$$f_t \ngtr f_{cto} = (f_{ctr}/\eta), \eta = 1.7$$

$$f_t = \frac{T}{A_c} = \frac{150*1000}{200*1000} = 0.75 \quad \text{MPa}$$

$$f_{ctr} = \frac{0.6\sqrt{f_{cu}}}{\eta} = \frac{0.6\sqrt{300}}{1.7} = 1.93 \text{ MPa}$$

$$f_t < f_{ctr} \qquad \text{(o.k)}$$

Sec. 2 $M_u = 100$ kN.m , $N_u = 0.0$ (water side sec.)

$$\therefore M = \frac{M_u}{1.5} = 66.7,\ t = \sqrt{\frac{M*1000}{3}} = \sqrt{\frac{6.67*1000}{3}} = 47.2 \text{ cm} \quad t = 50\sqrt{M} = 500 \text{ mm}$$

$$\cong 50 \text{ cm}$$

$$f_t > f_{ct}$$

$$\text{where } f_t = \frac{M}{Z} = \frac{6M}{bt^2} = \frac{66.7*10^6}{1000*\overline{500}^2} = 1.6 \text{ MPa}$$

$$t_v = t[1 + \frac{f_{ct}(N)^{=O}}{f_{ct}(M)}] = t = 50 \text{ cm}$$

$$\to \eta = 1.65$$

$$\therefore f_{ct} = \frac{0.6\sqrt{f_{cu}}}{\eta} = \frac{0.6\sqrt{30}}{1.65} = 1.99$$

$$\to f_t < f_{ct} \quad \text{o.k}$$

$$M_u = 100 \text{ kN.m} \ \& \ t = 500 \text{ mm} \to d = 460 \text{ mm}$$

$$d = c_1\sqrt{\frac{M_u}{f_{cu} b}}$$

$$460 = c_1\sqrt{\frac{100*10^6}{30*1000}} \to c_1 = 7.97$$

$$\therefore J = 0.826$$

$$\to A_s = \frac{M_u}{\beta_{cr} f_y d J}$$

- <u>use Φ10</u> $\to \beta_{cr} = 0.85$

$$A_s = \frac{100*10^6}{0.85*360*460*0.826} = 860 \text{ mm}^2 = 8.6 \text{ cm}^2$$

use 11 Φ 10 /m'

- <u>use Φ16</u> $\to \beta_{cr} = 0.75$

$$A_s = \frac{100*10^6}{0.75*360*460*0.826} = 975 \text{ mm}^2 = 9.75 \text{ cm}^2$$

use 5 Φ 16 /m'

or 9 Φ 12 /m'

$A'_s = 5$ Φ 10 /m'

<u>Sec. 3</u> $M_u = 100$ kN.m & $T_u = 10$ kN

$$M = \frac{100}{1.5} = 66.7 \; kN.m \quad \& \quad T = \frac{10}{1.5} = 6.7 \; kN$$

$$t = 50\sqrt{M} + 30 = 50\sqrt{66.7} + 30 = 438 \; mm \quad \text{take } t = 500 \, mm$$

$$f_t = f_{ct}(N) + f_{ct}(M)$$

$$f_{ct}(N) = \frac{T}{A_c} = \frac{6.7*1000}{1000*500} = 0.0134 \; N/mm^2$$

$$f_{ct}(M) = \frac{6M}{bt^2} = \frac{6*66.7*10^6}{1000*500^2} = 1.6 \; N/mm^2$$

$$f_t = 1.6 + 0.0134 = 1.6134 \; N/mm^2$$

$$f_{ct} = \frac{f_{ctr}}{\eta} \; , \; f_{ctr} = 0.6\sqrt{f_{cu}} = 3.28 \; N/mm^2$$

$$t_v = t[1 + \frac{f_{ct}(N)}{f_{ct}(M)} = 500[1 + \frac{0.0134}{16}] = 504.18 \; mm^2$$

$$\therefore \eta \cong 1.65$$

$$f_{ct} = \frac{3.28}{1.65} = 1.98 \; N/mm^2 > f_t \quad o.k$$

$$e = \frac{M_u}{T_u} = \frac{100}{10} = 10 \, m, \; \text{outside sec. (big ecc.)}$$

$$e_s = e - \frac{t}{2} + cover$$

$$= 10 - 0.25 + 0.04 = 9.79 \, m$$

$$M_{us} = T_u e_s = 10*9.79 = 97.9 \; kN.m$$

$$d = c_1\sqrt{\frac{M_{us}}{f_{cu} b}} \quad \therefore 460 = c_1\sqrt{\frac{97.9*10^6}{30*1000}} \rightarrow c_1 = 8.05$$

$$\therefore J = 0.826$$

use $\Phi_{max} = 16 \, mm \rightarrow \beta_{cr} = 0.75$

$$A_s = \frac{M_{us}}{\beta_{cr} f_y d J} + \frac{T_u}{\beta_{cr} f_y / \gamma_s}$$

$$= \frac{97.9*10^6}{0.75*360*0.826} + \frac{6.7*10^3*1.15}{0.75*360} = 983 \; mm^2$$

use 5 Φ 16 /m'

$A'_s = 5 \; \Phi \; 10$ /m'

Sec. 4 $M_u = 10$ kN.m & $T_u = 150$ kN

$$M = \frac{10}{1.5} = 6.7 \text{ kN.m} \quad \& \, T = \frac{150}{1.5} = 100 \text{ kN}$$

$$t = 50\sqrt{M} + 30$$

$$= 50\sqrt{6.7} + 30 = 159.4 \text{ mm}$$

$$0.6\,T = 0.6*100 = 60mm$$

$$\therefore \text{use } t = 200 \text{ mm}$$

$$f_t = f_{ct}(N) + f_{ct}(M) > f_{ctr}/\eta$$

$$f_{ct}(N) = \frac{T}{A_c} = \frac{100*1000}{1000*200} = 0.5 \text{ N/mm}^2$$

$$f_{ct}(M) = \frac{6M}{bt^2} = \frac{6*6.7*10^6}{1000*\overline{200}^2} = 1.005 \text{ N/mm}^2$$

$$t_v = t\left[1 + \frac{f_{ct}(N)}{f_{ct}(M)}\right] = 200\left[1 + \frac{0.5}{1.005}\right] = 299.5\,\text{mm}$$

$$\therefore \eta \cong 1.45 \,,\, f_{ctr} = 3.28 \text{ N/mm}^2$$

$$f_t = 0.5 + 1.005 = 1.505 \text{ N/mm}^2 \,,\, f_{ct} = \frac{3.28}{1.45} = 2.26 \text{ N/mm}^2$$

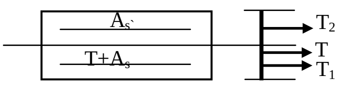
$$\therefore f_t < f_{ct} \quad \text{o.k}$$

$$M_u = 10 \text{ kN.m} \quad \& \quad T_u = 150 \text{ kN}$$

$$e = \frac{M_u}{T_u} = \frac{10}{150} = 0.067\,\text{m} < \frac{t}{2} - \text{cover}$$

$$T_1 = \frac{T_u}{2} + \frac{M_u}{d - d'} = \frac{150}{2} + \frac{10}{0.16} = 137.5 \text{ kN}$$

$$T_2 = T - T_1 = 150 - 137.5 = 12.5 \text{ kN}$$



$$\rightarrow \text{use } \phi\, 10 \quad , \beta_{cr} = 0.85$$

$$A_s = \frac{T_1}{\beta_{cr} f_y / \gamma_s} = \frac{137.5*1.15}{0.85*360} = 517 \text{ mm}^2$$

choose 7 ϕ 10 /m'

$$A'_s = \frac{T_2}{\beta_{cr} f_y / \gamma_s} = \frac{12.5*1.15}{0.85*360} = 47 \text{ mm}^2$$

use 5 ϕ 10/m'

2. Rectangular tanks with sliding Base (I)

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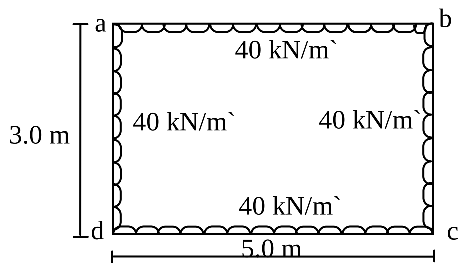
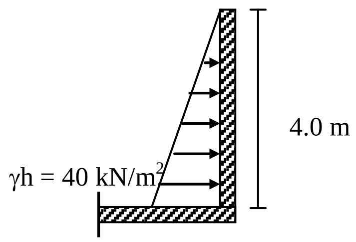
*** Consider 1.0 m Strip at lower end of wall.**





* Applying 3- moment Eqn to calculate the bending moment:

apply 3- moment Eqn at (a)

$M_d\ (3) + 2\ M_a\ (3+5) + M_b(5) = -6R_{el}$

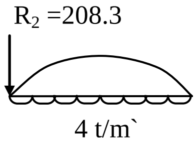
$R_e1 = -(20.83 + 4.50)\ 253.3$

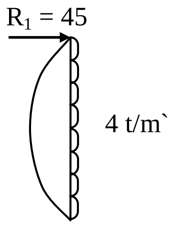
$M_a = M_b = M_d$

$3M_a + 16M_a + 5M_a = 6 * 253.3$

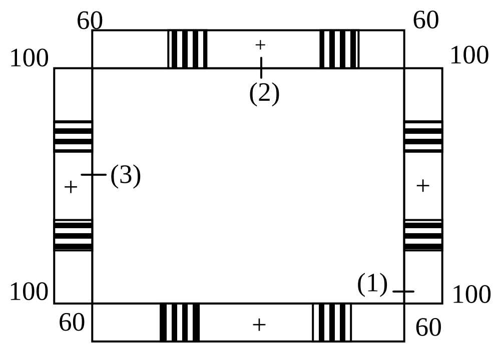
$24\ M_a = 1519.8$

$\therefore M_a = 630\ m.t$

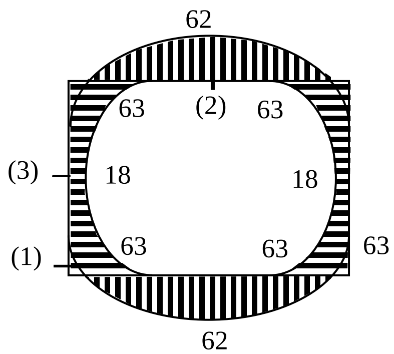




$$R = \frac{w\ell^3}{24}$$

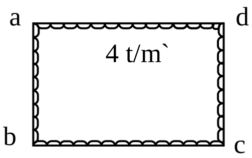


N. F. D



B. M. D

Calculation of moment by using method of moment distribution:



Joint	a		b		c		d	
member	ad	ab	ba	bc	cb	cd	dc	Da
R. S	0.33	0.2	0.2	0.33	0.33	0.2	0.2	0.33
D. F	0.62	0.38	0.38	0.62	0.62	0.38	0.38	0.62
F. E. M	30	83.3	+83.3	-30	+30	-83.3	+83.3	-30
	33	20.3			33	20.3		
			10.15	16.5			10.15	16.5
			-30.4	-49.6			-30.4	-49.6
	63	-63	63	-63	63	-63	+63	-63

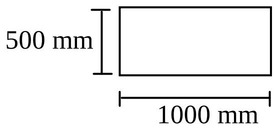
*** Design of Section (1):**

$t = 50\sqrt{M} + 30 = 50\sqrt{69.1} + 30 = 445.6$

$take \ \mathrm{t} = 500 \ \mathrm{mm}$

$\mathrm{A} = 1000 * 500 = 5 * 10^5 = 500 \ \mathrm{mm}$

$\mathrm{I} = \dfrac{1000 * 500^3}{12} = 1.042 * 10^{10} \ \mathrm{mm}^4$



$\mathrm{f_{ct}(N)} = \dfrac{100 * 10^3}{5 * 10^5} = 0.2$

$\mathrm{f_{ct}(M)} = \dfrac{63 * 10^6}{1.042 * 10^{10}} * 250 = 1.51 \ \mathrm{N/mm^2}$

$\mathrm{t_v} = \mathrm{t}(1 + \dfrac{\mathrm{f_{ct}}(N)}{f_{ct}(M)}) = 566.22 \ \mathrm{mm}$

$\therefore \dfrac{\mathrm{f_{ctr}}}{\eta} = \dfrac{0.6\sqrt{25}}{1.65} = 1.81 \ \mathrm{N/mm^2}$

$\mathrm{f_{ct}(N)} + \mathrm{f_{ct}(M)} = 1.71 < \dfrac{\mathrm{f_{ctr}}}{\eta} \ \mathrm{ok}$

M = 69.1 kN.m T = 100 kN , Water- side section

M_u = 94.5 kN.m T_u= 150 kN.m

$$e = \frac{M_u}{T_u} = 63.0cm > t/2$$

$$e_s = e - \frac{t}{2} + \text{cov}er = 0.45m$$

$$M_{us} = T.e_s = 67.5 \quad \text{kN.m}$$

$$d = c_1\sqrt{\frac{M_{us}}{f_{cu}B}}$$

$$430 = c_1\sqrt{\frac{67.5*10^6}{25*10}}$$

$$\therefore c_1 = 8.28 \qquad \text{j} = 0.826$$

$$\text{A}_s = \frac{M_{us}}{\beta_{cr} f_y jd} + \frac{T_u}{\beta_{cr} f_y/\gamma_s} = \frac{67.5*10^6}{0.85*360*0.826*430} + \frac{15*10^3}{0.85*360/1.15} = 1130 \ \text{mm}^2$$

$$= 11.3 \ \text{cm}^2$$

use 10 Φ 12 /m\`

Sec. Steel = 6 Φ 10 /m\`

* Design of section (2)

$M_u = 93$ kN.m , $T_u = 90$ kN t = 30 cm ; Air-side section

$$\text{e} = \frac{\text{M}_\text{u}}{\text{T}_\text{u}} = 1.03\,\text{m} > \frac{\text{t}}{2}$$

$$\text{e}_\text{s} = \text{e} - \frac{\text{t}}{2} + \text{cover} = 0.953$$

$$\text{M}_\text{us} = 85.8 \quad \text{kN.m}$$

$$\text{d} = \text{C}_1\sqrt{\frac{\text{M}_\text{us}}{\text{f}_\text{cu}B}} \qquad 230 = \text{C}_1\sqrt{\frac{85.8*10^6}{250*1000}}$$

$$\therefore \text{C}_1 = 3.93 \qquad \text{j} = 0.802$$

$$\text{A}_s = \frac{M_{us}}{f_y jd} + \frac{T_u}{f_y/\gamma_s} = \frac{85.8*10^6}{360*0.802*230} + \frac{90*10^3}{360/1.15}$$

$$= 20.85 \ \text{cm}^2$$

use 10 Φ 16 /m\`

Design of Section (3)

$M_u = 27$ kN.m $T_u = 150$ kN.m $t = 300$ mm , Water-side section

$A = 1000*300 = 3*10^5$ mm^2

$$I = \frac{1000*300^3}{12} = 2.25*10^9 \text{ mm}^4$$

$$f_{ct}(N) = \frac{100*10^3}{3*10^5} = 0.33 \text{ N/mm}^2$$

$$f_{ct}(M) = \frac{18*10^6}{2.25*10^9}*150 = 1.2$$

$$t_v = t(1+\frac{f_{ct}(N)}{f_{ct}(M)}) = 382.5 \text{ mm} \qquad \therefore \eta = 1.5$$

$$f_{ctr}/\eta = 2 \text{ N/mm}^2$$

$$f_{ct}(N) + f_{ct}(M) = 1.53 < \frac{f_{ctr}}{\eta} \qquad \text{ok}$$

$$e = \frac{M}{N} = 0.18 \; m > \frac{t}{2}$$

$$e_s = e - \frac{t}{2} + \text{cover} = 0.10 \text{ m}$$

$$M_{us} = 15 \text{ kN.m}$$

$$d = C_1\sqrt{\frac{M_{us}}{f_{cu}b}}$$

$$230 = C_1\sqrt{\frac{15*10^6}{25*1000}} \qquad \therefore C_1 = 9.39 \qquad j = 0.826$$

$$A_s = \frac{M_{us}}{\beta_{cr} f_y jd} + \frac{T_u}{\beta_{cr} f_y / \gamma_s}$$

$$= 8.22 \text{ cm}^2$$

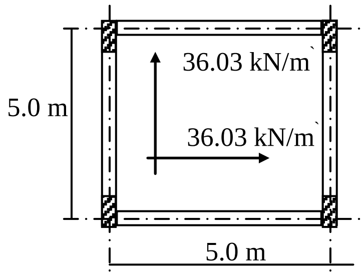
use 10 Φ 12 /m`

3. Rectangular tanks with Sliding Base (II)

3. Rectangular tanks with Sliding Base (II)

- **Prob. No. (1)** [Elevated tank with Sliding Base]
  Take thickness of base = 20 cm

- **Loads**
  w kN/m^2 = weight of covering + weight of water + weight of walls + o.w of base
  Weight of covering = 4 kN/m^2
  Weight of water = γ h = 40 kN/m^2
  Weight of walls : assume thickness of wall = 30 cm
  = 0.3*25.0*4* (5*2+4.40*2) = 564 kN
  Weight of walls /m^2 = 56.40/(5*5) = 2.256 t/m^2
  Weight of base = 0.2 *25 +0.5 = 5.5 kN/m^2
  w_{tot} t/m^2 = 4+ 40 +22.56 +5.5 = 72.06 kN/m^2

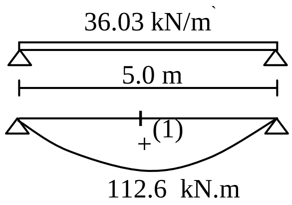


$$r = \frac{5.0}{5.0} = 1.0$$

$$\alpha = 0.5 \qquad \beta = 0.5$$

$$w_\alpha = w_\beta = 36.03 \ \ \text{kN/m}^2$$

- **Consider 1.0 m width:**



$$M_u = 169 \ \ \text{kN/m`} \qquad \text{(Air - side section)}$$

$$d = C_1 \sqrt{\frac{M_u}{f_{cu} B}}$$

$$160 = C_1 \sqrt{\frac{169*10^6}{25*1000}}$$

$$\therefore C_1 = 1.95 \qquad C_1 < C_{1\,min}$$

$$\text{take} \ \ t = 300 \, \text{mm}$$

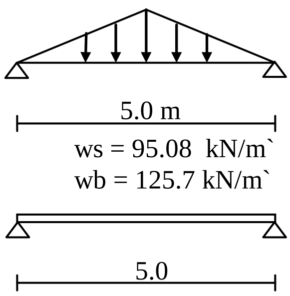
$$\therefore C_1 = 3.16 \qquad j = 0.757$$

$$A_s = \frac{M_u}{f_y jd} = \frac{169*10^6}{360*0.757*260} = 2385 \ \text{mm}^2/m = 23.85 \ \text{cm}^2/m$$

use 10 19 /m` in both directions bottom reinforcement

Use top reinforcement = 0.2 * 23.85 = 4.77 cm^2/m $\rightarrow$ Choose 7 10 /m`

Design of beams: assume beam 40 * 80 cm
O.w = 0.4 (0.8-0.30) *2.50= 0.50 t/m`
w_{slab}= 7.206 t/m^2
Equivalent load for shear
w_s= 0.50 + 7.206 *2.50*0.5 =9.508 t/m`
$Q_{u\,max}$ = 35.655 t



Equivalent load for flexure (bending)

w_b= 0.50+7.206*2.50*0.67= 12.57 t/m`

$M_{u\,max}$ = 58.92 m.t

$$d = C_1 \sqrt{\frac{M_u}{f_{cu} B}}$$

$$B = \textit{the} \text{ least of } \begin{cases} 6\,t_s + b = 220 \text{ cm} \\ \frac{L_2}{10} + b = 90 \text{ cm} \quad \text{(controls)} \\ \frac{1}{2} C.L \rightarrow C.L = 250 \text{ cm} \end{cases}$$

$$\therefore 75 = C_1 \sqrt{\frac{58.92 * 10^5}{250 * 90}} \qquad \therefore C_1 = 4.63 \qquad j = 0.821$$

$$A_s = \frac{M_u}{f_y\, jd} = 26.58 \text{ cm}^2 \qquad \text{use } 7\ \Phi\ 22 \qquad A_s' = 3\ \Phi 19$$

<u>check of shear</u>

$$q_u = \frac{Q_u}{bd} = \frac{35.655 * 10^3}{40 * 75} = 11.885 \ kg/cm^2 > q_{u_c} = 9.68 \text{ kg/cm}^2 \quad \text{unsafe}$$

use 7 ϕ 8/m` 4 branches

$$q_{u_{act}} = q_{u_s} + q_{u_c}/2 = 15.79 > 11.885 \text{ kg/cm}^2 \qquad \therefore \text{O.K.}$$

Prob. No. (2) [Resting on soil tank with sliding base]

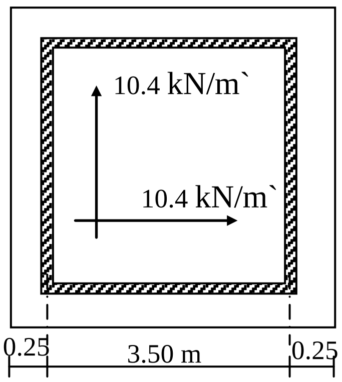
- **Check of bearing capacity:**

Assume t = 250 mm
$\sum$ vl load = wall load + base load + weight of water
(assume t_{wall} = 300 mm)
Weight of walls = 0.3*25*3 (2*4+2*3.40) =33.30 t
w t/m^2= 333 / (4*4) = 20.8 kN/m^2

- Wall load = 20.8 kN/m^2
- Base load = 0.25 *25 = 6.25 t/m^2 +0.5 (flooring) = 6.75 kN/m^2
- Weight of water = γh = 10 *3 = 30 kN/m^2

Total load = 57.55 kN/m^2 < 100 kN/m^2 ok

Loads

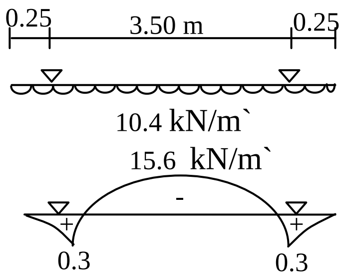


w kN/m^2= wt of walls (assume t_{wall} = 300 mm)
w kN/m` = 333 / (4*4) = 20.8
r = 1.0 $\alpha = \beta = 0.5$
$W_\alpha = W_\beta = 10.4$ kN/m^2

Consider 1.0 m width

$M_{u\,max}$ = 23.4 kN.m (Water-side section)

Water side



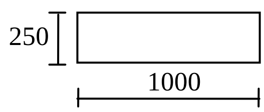
$$t = 50\sqrt{M}$$

$$= 50\sqrt{15.6} = 197.4 \text{ mm}$$

take t = 250 mm

$$A = 1000 * 250 = 25000 \text{ mm}^2$$

$$z = \frac{bt^2}{6} = \frac{1000 * 250^2}{6} = 10.42 * 10^6$$



$$f_{ct}(N) = 0$$

$$f_{ct}(M) = \frac{M}{Z} = \frac{15.6 * 10^6}{10.42 * 10^6} = 1.49 \text{ N/mm}^2$$

t_v = 250 mm $\qquad f_{ctr}/\eta = 2.2$

$$f_{ct}(M) = 1.498 < \frac{f_{ctr}}{\eta} \qquad \text{ok}$$

$$d = C_1 \sqrt{\frac{M_u}{f_{cu} B}}$$

$$200 = C_1 \sqrt{\frac{23.4*10^6}{25*1000}}$$

$$\therefore C_1 = 6.54 \qquad \mathrm{j} = 0.826$$

$$\mathrm{A_s} = \frac{\mathrm{M_u}}{\beta_{\mathrm{cr}} \mathrm{f_y} jd} = \frac{23.4*10^6}{0.85*360*200} = 463 \ \mathrm{mm}^2$$

use 7 Φ 10 /m` (Top reinforcement)

Use bottom reinforcement 5 Φ 10 /m`

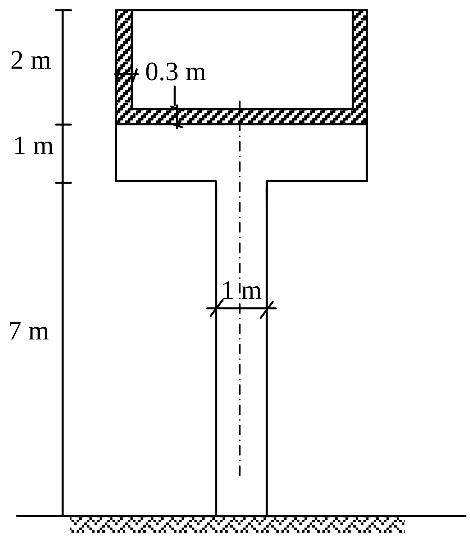
Note that in this case, walls of the tank act as supporting beam with adequate capacity.

4. Elevated Conduits

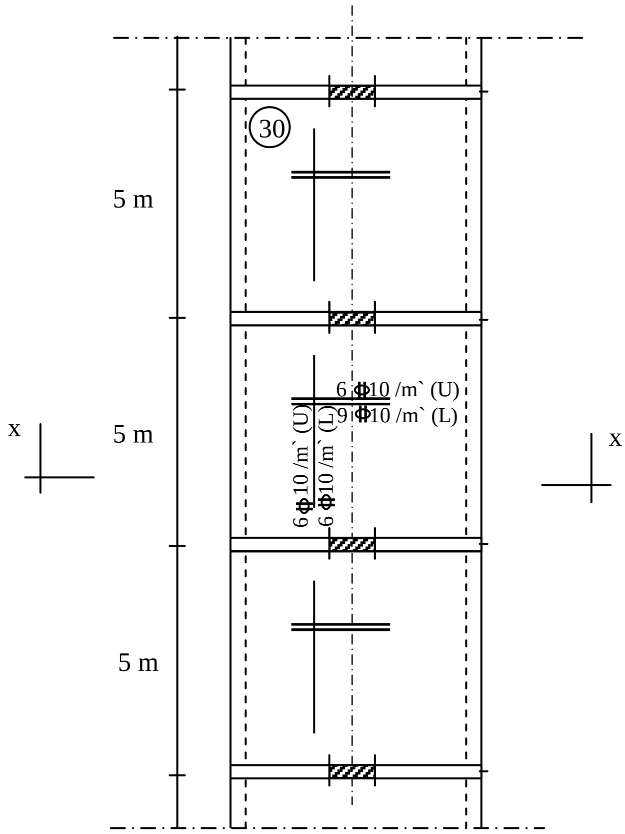
4. Elevated Conduits

(1)

(i)



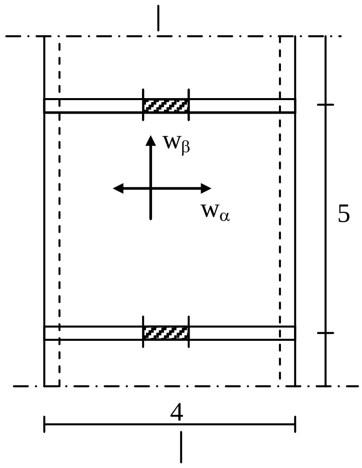
Sec. Elevation x-x



Plan

(ii) Acting load

assume t_s = 30 cm



- Walls:
  the wall will be considered as
  one way slab in vertical direction.

  p= γ h = 1 * 17 = 17 kN/m^2

- Floor:

$$w = \gamma\ t_s + \gamma_w h$$

$$= 25 * 0.3 + 1 * 17 = 24.5\ \text{kN/m}^2$$

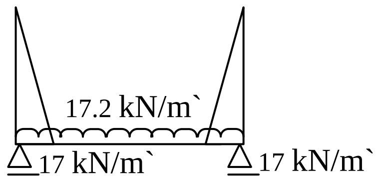
$$r = \frac{5 * 0.76}{4 * 0.76} = 1.25$$

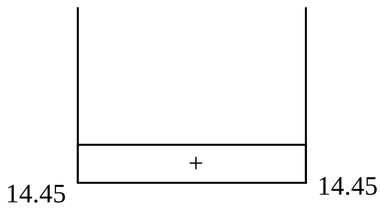
$$\alpha = \frac{r^4}{r^4 + 1} = 0.7\ \rightarrow \beta = 1 - \alpha = 0.3\ \text{(Grashoff 's coefficients)}$$

$$w_\alpha = 0.7 * 24.5 = 17.15\ \text{kN/m}^2\ , \qquad w_\beta = 0.3 * 24.5 = 7.4\ \text{kN/m}^2$$

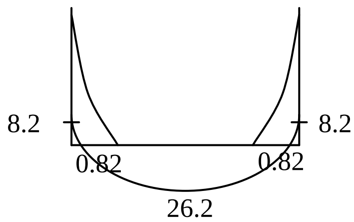
Straining Actions

* Vertical strip



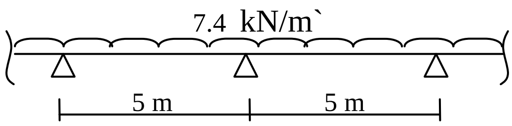


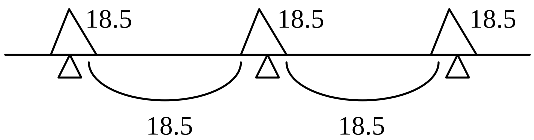
N. F. D



B. M. D

* Section in Floor

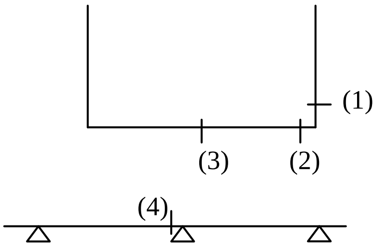




B. M. D

Design of critical sections

Consider $f_{cu} = 25$ kN/cm^2
St. 360/520



Sec. (1)

water side section

$$M = 8.2 \quad kN.m$$

$$t = 50\sqrt{M} = 50\sqrt{8.2} = 143.1 \ mm$$

take $t_s = 300$ mm

$$f_{ct} = f_{ct}(M) > \frac{f_{ctr}}{\eta}$$

$$\text{where, } f_{ct}(M) = \frac{6M}{bt^2} = \frac{6*8.2*10^6}{1000*\overline{300}^2} = 0.546 \quad N/mm^2$$

$$f_{ctr} = 0.6\sqrt{f_{cu}} = 0.6\sqrt{25} = 3 \ N/mm^2$$

$$t_v = t[1+\frac{f_{ct}(N)}{f_{ct}(M)}] = 300 \rightarrow \eta = 1.45$$

$$\therefore \frac{f_{ctr}}{\eta} = \frac{29.76}{1.45} = 2.06 \ N/mm^2 > f_{ct} \qquad (o.k)$$

$$M_{\bar{u}} = 1.5*8.2 = 12.3 \ kN.m$$

$$t = 300 \, mm \rightarrow d = 260 \, mm$$

$$d = c_1\sqrt{\frac{12.3*10^6}{25*1000}} \rightarrow c_1 = 11.7$$

$$\rightarrow J = 0.826$$

$$\therefore \text{use } \phi_{max} = 10mm \rightarrow \beta_{cr} = 0.85$$

$$A_s = \frac{M_u}{\beta_{cr} f_y dJ} = \frac{12.3*10^6}{360*260*0.826} = 187 \ mm^2 = 1.87 \, cm^2$$

choose 6 ϕ 10 /m`

A`$_s$ = 6ϕ10 /m`

Sec. (2)

$M = 8.2$ kN.m & $T = 14.45$ kN (water side section)

$$t = 50\sqrt{M} + 30 = 173 \text{ mm}$$

take $t_s = 300$ mm

$$f_{ct} = f_{ct}(N) + f_{ct}(M) > \frac{f_{ctr}}{\eta}$$

$$f_{ct}(N) = \frac{T}{A_c} = \frac{14.45*1000}{300*1000} = 0.048 \text{ kN/mm}^2$$

$$f_{ct}(M) = \frac{6M}{bt^2} = \frac{6*8.2*10^6}{1000*\overline{300}^2} = 0.546 \text{ kN/mm}^2$$

$$\therefore f_{ct} = 0.048 + 0.546 = 0.594 \text{ kN/mm}^2 \rightarrow$$

$$t_v = t[1 + \frac{f_{ct}(N)}{f_{ct}(M)}] = 300[1 + \frac{0.048}{0.547}] = 326.3 \text{ mm}$$

$$\therefore \eta = 1.45 \text{ , } f_{ctr} = 3 \text{ kN/mm}^2 \quad \therefore \frac{f_{ctr}}{\eta} = 2.06$$

$$\therefore f_{ct} < \frac{f_{ctr}}{\eta} \qquad \text{o.k}$$

$M_u = 1.5*8.2 = 12.3$ kN.m & $T_u = 1.5*14.45 = 21.7$ kN

$$e = \frac{M_u}{T_u} = \frac{12.3}{21.7} = 0.57\,m > \frac{t}{2} \quad \text{(big ecc.)}$$

$$e_s = e - \frac{t}{2} + cover = 0.57 - 0.15 + 0.04 = 0.46\,m$$

$$M_{us} = T_u e_s = 21.7*0.46 = 10 \quad kN.m$$

$$d = c_1\sqrt{\frac{M_{us}}{f_{cu}b}} \qquad \therefore 260 = c_1\sqrt{\frac{10*10^6}{25*1000}} \rightarrow c_1 = 13$$

$$J = 0.826$$

use $\phi_{max} = 10\,mm \qquad \therefore \beta_{cr} = 0.85$

$$A_s = \frac{M_{us}}{\beta_{cr} f_y dJ} + \frac{T_u}{\beta_{cr} f_y/\gamma_s}$$

$$= \frac{10*10^6}{0.85*360*260*0.826} + \frac{21.7*1000*1.15}{0.85*360}$$

$$= 234 \ mm$$

choose 6 Φ 10 /m`

$A`_s = 0.2\,A_s$ use 6Φ10 /m`

Sec. (3)

M =26.2 kN.m & T = 14.45 kN (air side section)
t = 300 mm & d = 260 mm

$M_u = 1.5*26.2 = 39.3$ kN.m & $T_u = 1.5*14.45 = 2.17$ kN

$$e = \frac{M_u}{T_u} = \frac{39.3}{21.7} = 1.81\,m \quad \text{(big ecc.)}$$

$$e_s = e - \frac{t}{2} + cover = 1.81 - 0.15 + 0.04 = 1.7\,m$$

$$M_{us} = T_u e_s = 21.7*1.7 = 36.9 \quad kN.m$$

$$d = c_1\sqrt{\frac{M_{us}}{f_{cu}b}} \qquad \therefore 260 = c_1\sqrt{\frac{36.9*10^6}{25*1000}} \rightarrow c_1 = 6.77$$

$$J = 0.826$$

use $\phi_{max} = 10\,mm$

$$A_s = \frac{M_{us}}{f_y dJ} + \frac{T_u}{f_y/\gamma_s}$$

$$= \frac{36.9*10^6}{360*260*0.826} + \frac{21.7*1000}{360/1.15} = 200\,mm^2$$

choose 6 10 /m`

$A`_s = 6$ 10 /m`

Sec. (4)

$M = 18.5$ kN.m (water side section)

$t = 50\sqrt{M} = 50\sqrt{18.5} = 215$ mm

take $t = 300$ mm

$f_{ct} = f_{ct}(M) > \frac{f_{ctr}}{\eta}$

$f_{ct}(M) = \frac{6M}{bt^2} = \frac{6*18.5*10^6}{1000*\overline{300}^2} = 1.23$ N/mm^2

$t_v = t[1 + \frac{f_{ct}(N)}{f_{ct}(M)}] = t = 300\,mm \rightarrow \eta = 1.45$

$f_{ctr} = 0.6\sqrt{f_{cu}} = 3$ N/mm^2 $\rightarrow \frac{f_{ctr}}{\eta} = 2.06$ N/mm^2

$\therefore f_{ct} < \frac{f_{ctr}}{\eta}$ o.k

$d = c_1\sqrt{\frac{M_{us}}{f_{cu}b}}$ $\therefore 26 = c_1\sqrt{\frac{18.5*10^6*1.5}{25*1000}} \rightarrow c_1 = 7.8$

$J = 0.826$

use $\phi_{max} = 10$ mm $\therefore \beta_{cr} = 0.85$

$A_s = \frac{M_{us}}{\beta_{cr} f_y dJ} = \frac{1.5*18.5*10^6}{0.85*360*260*0.826} = 422.3$ mm^2

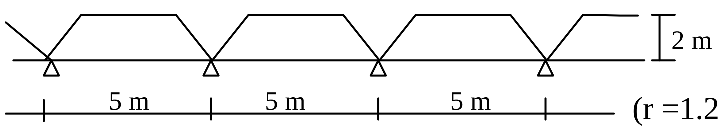
choose 6 Φ 10 /m`

$A`_s = 6$ Φ 10 /m`

Design of wall as beam

o.w= γ bt = 25 *0.3 *2 = 15 kN/m`

w_f = 24.5 kN/m^2



(r =1.25 , α = 0.79 , β = 0.6)

Load for shear:

w = o.w + β w_f x = 1.5+0.6*24.5*2 = 44.4 kN/m`

Load for moment

w = o.w + α w_f x = 1.5+0.79*2.45*2 = 5.37 t/m`

L_{eff}= C.L-C.L= 5 m

= 1.05 l_n= 1.05 *4.7 = 4.94 m →

∴ t/L_{eff} = 200/494 = 0.405 > 0.4 (deep beam)

$$M^{+ve} = \frac{wl^2}{12} = \frac{53.7*5^2}{12} = 112\ k\text{N/m}, M^{-ve} = \frac{wl^2}{10} = \frac{53.7*5^2}{10} = 134\ k\text{N/m`}$$

For - ve moment

$$M_u = 1.5*134 = 201\ k\text{N.m},\ d = 1900\ \text{mm}$$

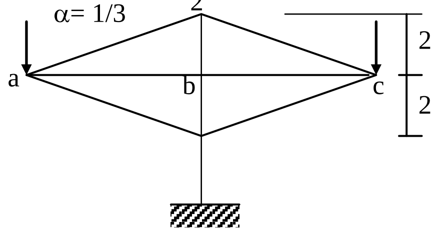
$$d = c_1\sqrt{\frac{M_u}{f_{cu}b}} \qquad ,1900 = c_1\sqrt{\frac{201*10^6}{25*300}} \rightarrow c_1 = 11.6$$

$$J = 0.826$$

$$A_s = \frac{M_u}{f_y dJ} = \frac{201*10^6}{360*1900*0.826} = 360\ \text{mm}^2$$

use 3 Φ 16 for - ve &+ ve moment /m`

* Shear capacity has to be checked according to code provisions for deep beams



Max. reaction of beam

R = 1.1 wl = 1.1 *44.4 *5 = 244 kN

For beam

Sec. of beam = 300 * 1000

o.w = 25 *0.3*1= 8 kN/m`

Load for moment w = o.w +α w x = 8+2*1/3 *24.5*2= 41 kN/m`

Load for shear w = o.w +β w x = 8+2*1/2 *24.5*2= 57 kN/m`

$$M = 41*2^2/2 = 82\ \text{kN.m}$$

$$M_u = 1.5\ M = 123\ \text{kN.m}$$

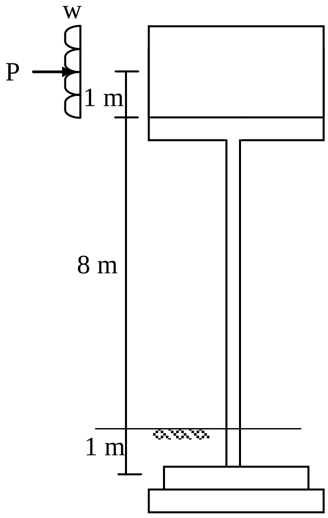
$$d = c_1\sqrt{\frac{M_u}{f_{cu}b}} \qquad ,95 = c_1\sqrt{\frac{123*10^6}{25*300}} \rightarrow c_1 = 7.4$$

$$J = 0.826$$

$$A_s = \frac{M_u}{f_y dJ} = \frac{123*10^6}{360*950*0.826} = 440\ \text{mm}^2$$

use 3 16 , $A`_s$ = 2 12

(iii) loads on columns



vl. Load = 2 [244 + 57 *2]=716 kN
N = 716 *1.1 = 788 kN

W_{wind} = 716 *5 = 5 kN/m`
P= 5 * 2 = 10 kN
M = 10*10 = 100 kN.m

ϕ ϕ

Design of sec.
N = 788 kN
N_u = 1.5 *788 = 1182 kN

Design for N only
$N_u = 0.35\ A_c\ f_{cu} + 0.67\ f_y\ A_{sc}$
$N_u = A_c\ [0.35\ f_{cu} + 0.67\ f_y\ \mu]$
$1182*1000 = A_c\ [0.35*25+0.67*360*1/100]$
$A_c = 1059\ cm^2$
Take column 300 *400 mm
To sustain the effect of buckling, take column 300 * 1000
$H_e = k\ H_o$ = 2.2 *8 = 17.6 m
Consider unbraced column

$$\lambda = \frac{H_e}{t} = \frac{17.6}{1} = 17.6 < \lambda_{max} \quad \text{o.k}$$

$$\lambda > 10 \quad \therefore \text{(long column)}$$

$$\delta = \frac{\lambda^2 t}{2000} = \frac{\overline{17.6}^2 * 1000}{2000} = 155 \text{ mm}$$

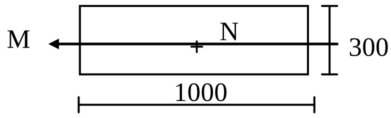
$$\therefore M_{add} = P\delta = 788 * \frac{155}{1000} = 12.2 \text{ m.t}$$

$M_{design} = M_2 + M_{add}$, where M_2 is due to wind load

$$= 100 + 122 = 222 \text{ kN.m}$$

$N_u = 1182$ kN , $M_u = 1.5 * 222 = 333$ kN.m

$$e = \frac{M_u}{N_u} = \frac{333}{1182} = 0.3 \text{ m}$$



$$\frac{e}{t} = \frac{0.3}{1} = 0.3 > 0.05 \quad \text{o.k}$$

use interaction diagram for $f_y = 3600$, $\alpha = 1$, $\xi = 0.8$

$$k = \frac{N_u}{f_{cu} bt} = \frac{1182 * 1000}{25 * 300 * 1000} = 0.16$$

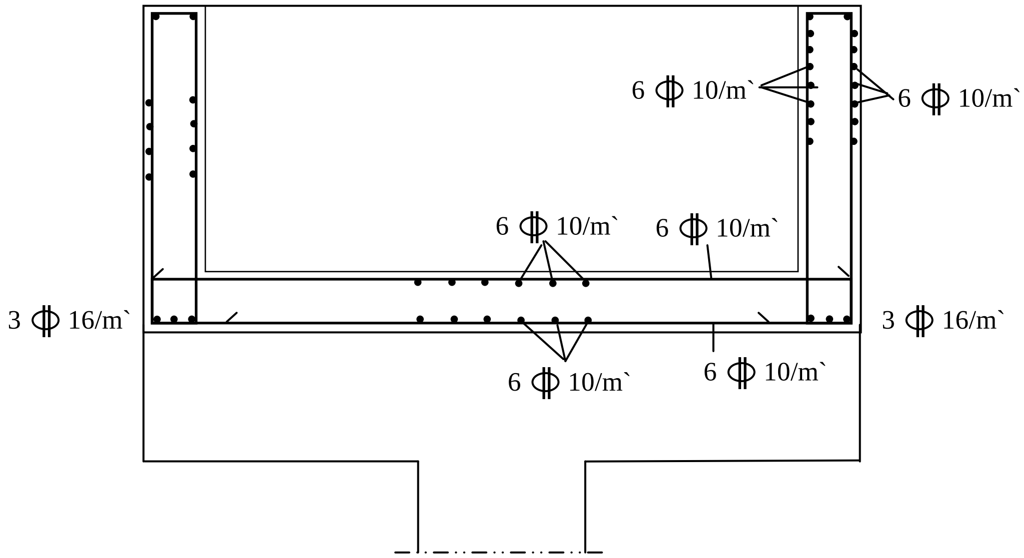
$$k\frac{e}{t} = \frac{M_u}{f_{cu} bt^2} = \frac{333 * 10^6}{25 * 300 * \overline{1000}^2} = 0.04$$

$\rho = 0.0$

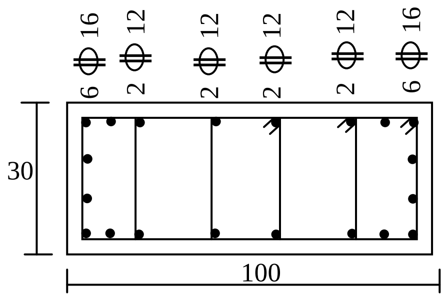
use min steel $= \frac{0.4}{100}$ bt /side

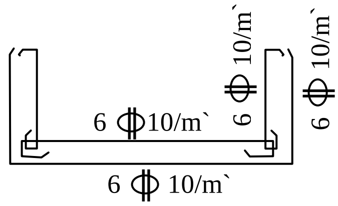
$$= \frac{0.4}{100} * 300 * 1000 = 1200 \quad \text{mm}^2$$

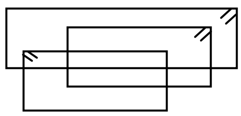
choose 6 Φ 16



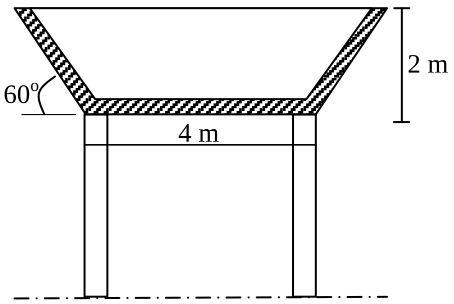
Cross sec. Of column



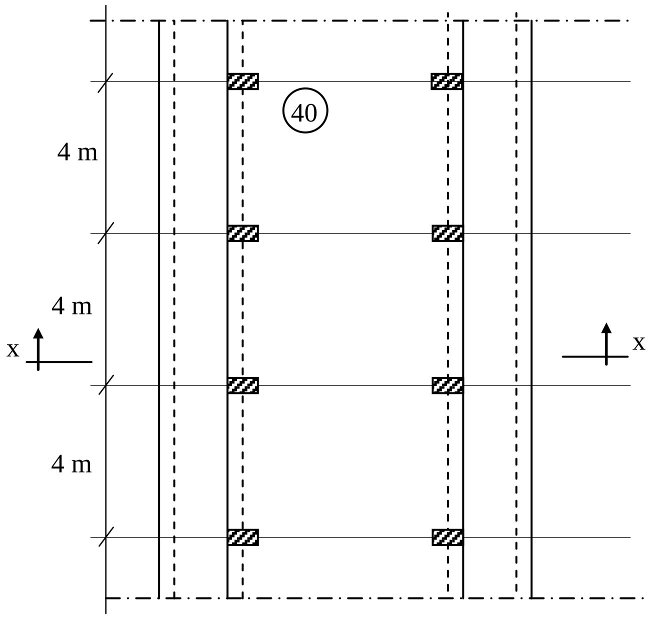




Prob. (2)



Sec. Elevation x-x



Plan

Loads

The floor and wall will be considered as way slab in the plane of the paper.

Assume t = 40 cm

Floor

$w_f = \gamma\ t_s + \gamma_w\ h$

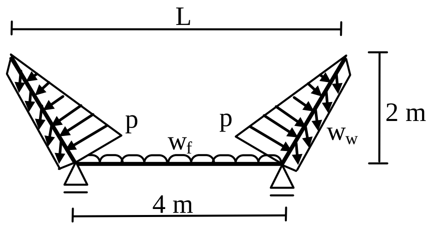
$= 25 * 0.4 + 10*2 = 30\ \ kN/m^2$

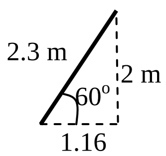
Wall

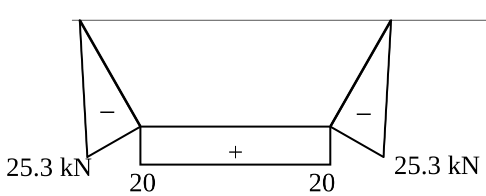
$w_w = \gamma\ t_s = 25 * 0.4 = 10\ \ kN/m^2$

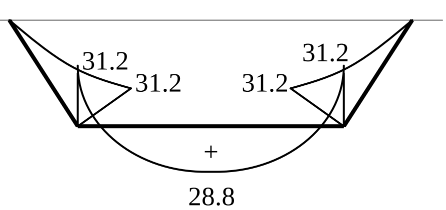
$P = \gamma_w\ h = 20\ \ kN/m^2$

Straining Actions

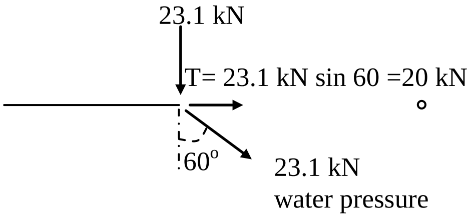




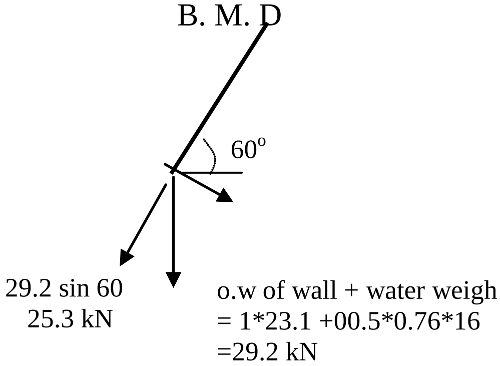




N. F. D

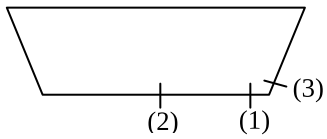


B. M. D



Design of critical sections

Sec. (1)



$M = 31.2 \text{ kN.m} \quad , \text{ T} = 20 \text{ kN} \text{ (water side section)}$

$\text{t} = 50\sqrt{M} + 30 = 50\sqrt{31.2} + 30 = 309 \text{ mm}$

$\text{take} \quad \text{t} = 400 \text{ mm}$

$f_{ct} = f_{ct}(N) + f_{ct}(M) > \frac{f_{ctr}}{\eta}$

$f_{ct}(N) = \frac{T}{A_c} = \frac{20*1000}{100*40} = 0.5 \text{ N/mm}^2$

$f_{ct}(M) = \frac{6M}{bt^2} = \frac{6*312*10^6}{1000*\overline{400}^2} = 1.17 \text{ N/mm}^2$

$f_{ct} = 0.5 + 1.17 = 1.67 \text{ N/mm}^2$

$t_v = t[1 + \frac{f_{ct}(N)}{f_{ct}(M)}] = 400[1 + \frac{0.5}{1.17}] = 570 \text{ mm} \text{ , } \eta = 1.65$

$f_{ctr} = 0.6\sqrt{f_{cu}} = 3 \text{ N/mm}^2 \rightarrow \frac{f_{ctr}}{\eta} = 1.82 \text{ N/mm}^2$

$\therefore f_{ct} > \frac{f_{ctr}}{\eta} \qquad \text{o.k}$

$M_u = 1.5*31.2 = 46.8 \text{ kN.m} \quad \& \quad T_u = 1.5*20 = 30 \text{ kN}$

$e = \frac{M_u}{T_u} = \frac{46.8}{30} = 1.56\text{m} > \frac{t}{2} \quad \text{(big ecc.)}$

$e_s = e - \frac{t}{2} + \text{cover}$

$= 1.56 - 0.2 + 0.04 = 1.40 \text{ m}$

$M_{us} = T_u e_s = 30*1.4 = 42 \text{ kN.m}$

$d = c_1\sqrt{\dfrac{M_u}{f_{cu}b}}$, $360 = c_1\sqrt{\dfrac{42*10^6}{25*1000}} \rightarrow c_1 = 8.78$

$J = 0.826$

use $\phi_{max} = 10$ mm $\therefore \beta_{cr} = 0.85$

$$A_s = \frac{M_{us}}{\beta_{cr} f_y d J} + \frac{T_u}{\beta_{cr} f_y / \gamma_s}$$

$$= \frac{42*10^6}{0.85*360*360*0.826} + \frac{30*1000}{0.85*360/1.15} = 5.74 \ mm^2$$

choose 8 Φ 10 /m\`

A\`$_s$ = 6 Φ 10 /m\`

Sec. (2)

$M = 288$ kN.m & T= 20 kN (air side section)

$M_u = 1.5*288 = 43.2$ m.t & $T_u = 1.5*20 = 30$ kN

$e = \dfrac{M_u}{T_u} = \dfrac{43.2}{30} = 43.2$ kN.m $> \dfrac{t}{2}$ (big ecc.)

$e_s = e - \dfrac{t}{2} + cover = 1.44 - 0.2 + 0.04 = 1.28$ m

$M_{us} = T_u e_s = 30*1.28 = 38.4$ kN.m

$d = c_1\sqrt{\dfrac{M_{us}}{f_{cu}b}}$, $360 = c_1\sqrt{\dfrac{38.4*10^6}{25*1000}} \rightarrow c_1 = 9.19$

Φ

Φ

$J = 0.826$

use $\phi_{max} = 10$ mm $\therefore \beta_{cr} = 0.85$

$$A_s = \frac{M_{us}}{f_y d J} + \frac{T_u}{f_y / \gamma_s}$$

$$= \frac{38.4*10^6}{360*360*0.826} + \frac{30*1000}{360/1.15} = 455 \ mm^2$$

choose 6 10 /m\`

A\`$_s$ = 6 10 /m\`

Sec. (3)

$M = 31.2$ kN.m , N = - 25.3 kN (water side section)

Big. Ecc. compression

$t = 50\sqrt{M} - 30 \cong 249$ mm take t = 400 mm

$f_{ct} = f_{ct}(N) + f_{ct}(M)$

$f_{ct}(N) = \frac{-25.3*1000}{1000*400} = -0.063$ kN/mm²

$f_{ct}(M) = \frac{6M}{bt^2} = \frac{6*31.2*10^6}{1000*400^2} = 1.17$ N/mm²

$f_{ct} = -0.063 + 1.17 = 1.107$ N/mm²

$t_v = t[1 + \frac{f_{ct}(N)}{f_{ct}(M)}] = 400[1 - \frac{0.063}{1.17}] = 421$ mm , $\eta = 1.6$

$f_{ctr} = 0.6\sqrt{f_{cu}} = 3$ N/mm²

$\therefore \frac{f_{ctr}}{\eta} = 1.86$ N/mm² $> f_{ct}$ (o.k)

$M_u = 1.5*31.2 = 46.8$ kN.m

$N_u = -1.5*2.53 = -38$ kN

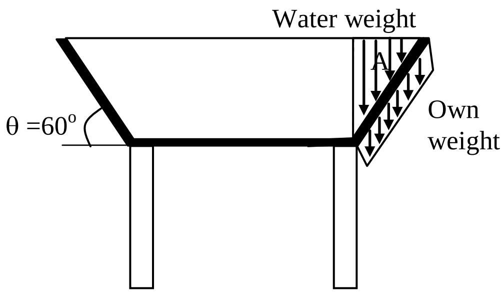
$\therefore$ Use $\phi_{max} = 10$ mm $\therefore \beta_{cr} = 0.85$

$\therefore \beta_{cr} f_y = 0.85*360 = 306$ N/mm²

$\Rightarrow$ Use strength interaction diagram for $f_y = 2800$ kg/cm²,

$\xi = 0.8$, $\alpha = 1$ $\Rightarrow A_s = A`_s = 8\ \Phi\ 10$ /m` (same as for sec.1)

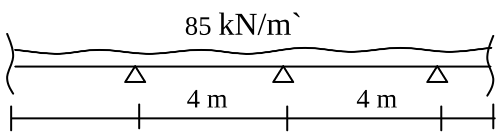
*** Design of wall as deep beam**



* Loads in the plan of the wall

- o.w of wall = γ b t sin θ
  = 25 * 0.3 *2.31 sin 60
  = 15 kN/m`
- wt of water = γ_w A sin θ
  = 10 * ½ *1.16 *2 sin 60
  = 10 kN/m`
- Load from floor = w_f *2 = 30* 2 = 60 kN/m`

$\therefore w_t$ = 15 +10 +60 = 85 kN/m`



$$M_{max}^{+ve} = \frac{wl^2}{12} = \frac{85*4^2}{12} = 113.3 \text{ kN/m}$$

$$M_{max}^{-ve} = \frac{wl^2}{10} = \frac{85*4^2}{10} = 136 \text{ kN/m}$$

Check of deep beam condition

$$L_{eff} = C.L - C.L = 4m$$

$$= 1.05\,L_n = 1.05*3.7 = 3.89 \text{ m}$$

$$\frac{t}{L_{eff}} = \frac{2.31}{3.89} = 0.59 > 0.4 \quad \text{Deep beam}$$

$$\text{for - ve moment} \qquad t = 2310 \text{ mm} \rightarrow d = 2200 \text{ mm}$$

$$d = c_1\sqrt{\frac{M_u}{f_{cu}b}}$$

$$2200 = c_1\sqrt{\frac{1.5*136*10^6}{25*300}}$$

$$C_1 = 13.34 \rightarrow J = 0.826$$

$$A_s = \frac{M_u}{f_y dJ} = \frac{1.5*136*10^6}{360*2200*0.826} = 312 \text{ mm}^2$$

$$A_{s\,min} = greater \begin{cases} \text{Smaller} \begin{cases} 1.3*312 = 406 \text{ mm}^2 \\ \frac{11}{360}*300*2200 = 2017 \text{ mm}^2 \end{cases} \\ 0.15/100*30*2200 = 990 \text{ mm}^2 \end{cases}$$

$\therefore use\ A_s = 5\Phi16$ for both negative and positive momnts

- Shear capacity has to be checked according to code provisions

(iii) Design of column

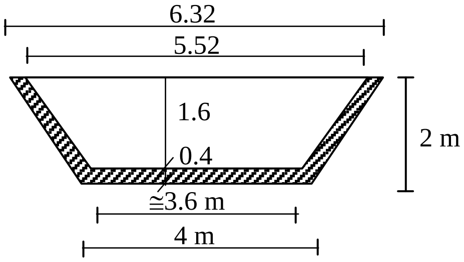
Load on each column

concrete

$$\frac{1}{2}*4[(\frac{6.32+4}{2})*2-(\frac{5.52+3.6}{2})*1.6]*25$$

water

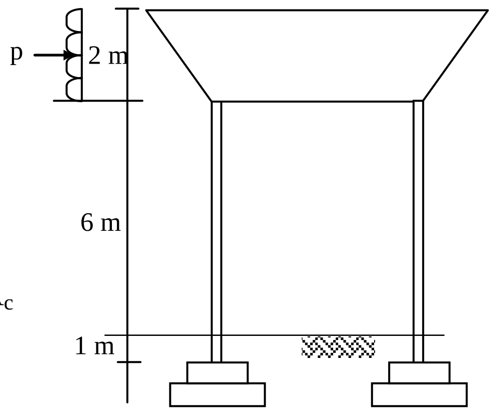
$$+\frac{1}{2}*4[(\frac{5.52+3.6}{2})*1.6]*10 = 295.9$$



$$\textit{due}\text{ to o.w of column} \rightarrow P_{column} = 1.1*295.9 = 325.2\ \text{kN}$$

Wind load

P = 1 * 2 * 4 = 8 kN
M = 8 * 8 = 64 kN
$\therefore$M/column = 64/2 = 32 kN.m



Design for N only

N_u = 1.5 * 325.5 = 488.3 kN
$N_u = [0.35\ f_{cut} + 0.67\ f_y\ \mu]\ A_c$
$488.3 * 1000 = [0.35 *25 + 0.67 *360 * 1/100]\ A_c$
$A_c = 43700\ mm^2$

Take sec. 300 * 500

unbraced column

$$\lambda = \frac{H_e}{t}\ , H_e = k\ H_o = 1.6*7 = 11.2m$$

$$\lambda = \frac{11.2\ *100}{50} = 22.4 < (\lambda_{max} = 23)$$

$$\therefore \text{take}\ \ t = 70 \rightarrow \lambda = 16 > 10 \quad \text{(long column)}$$

$$\therefore \delta = \frac{\lambda^2 t}{2000} = \frac{16^2 * 700}{2000} = 89.6\ \text{mm}$$

$$M_{add} = P.\delta = 325.5*\frac{89.6}{1000} = 29 \quad \text{kN.m}$$

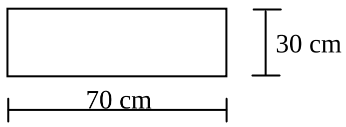
$$M_{design} = 32+29 = 61\,k\text{N.m}$$

$$\therefore M_u = 1.5*61 = 92 \quad \text{kN.m}$$

* Design for M & N

$M_u = 9.2$ m.t

$N_u = 48.83$ t



$$e = \frac{M_u}{N_u} = \frac{92}{488.3} = 0.2\text{m}$$

$$\frac{e}{t} = \frac{0.2}{0.7} = 0.3 > 0.05$$

use interaction diagram for $f_y = 360, \alpha = 1, \xi = 0.8$

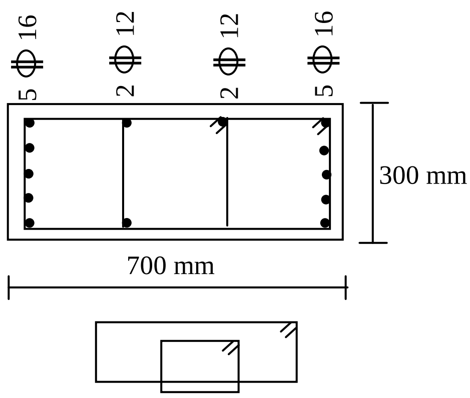
$$k = \frac{N_u}{f_{cu}bt} = \frac{488.3*1000}{25*300*700} = 0.093$$

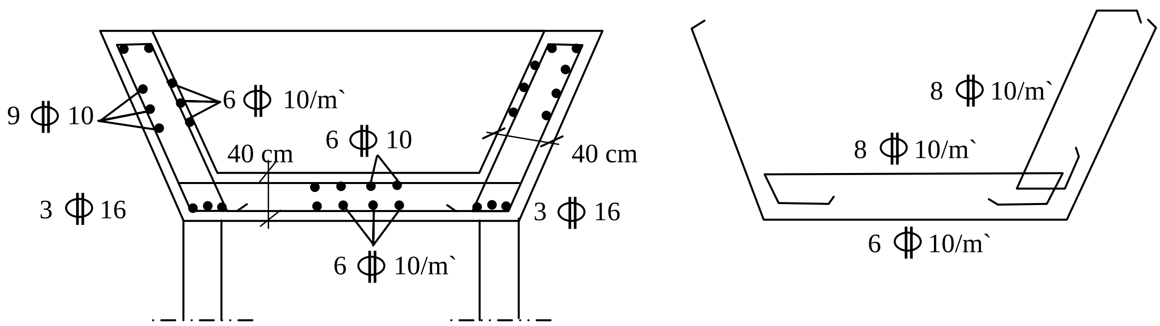
$$k\frac{e}{t} = \frac{M_u}{f_{cu}bt^2} = \frac{92*10^6}{25*300*700^2} = 0.025, \quad \rho = o$$

$$\text{use min. steel} \quad A_s = \frac{0.4}{100}*300*700 = 8400 \ \text{mm}^2/\text{side}$$

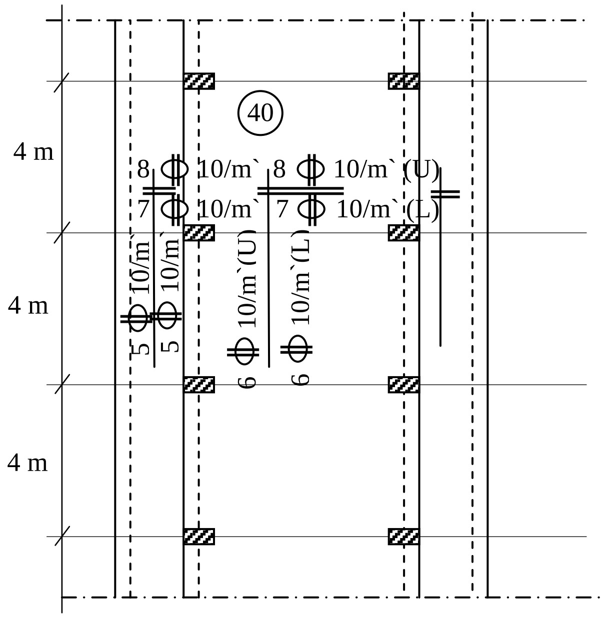
use 5 Φ 16

Sec. in column





Cross-section of tank


40
4 m
4 m
4 m
8 Φ 10/m` 8 Φ 10/m` (U)
7 Φ 10/m` 7 Φ 10/m` (L)
5 Φ 10/m`
5 Φ 10/m`
6 Φ 10/m`(U)
6 Φ 10/m`(L)

Plan

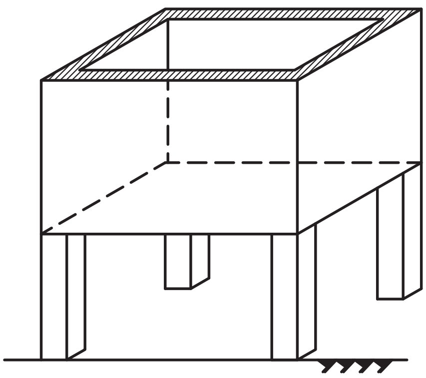
Water tanks

خزانات المياه

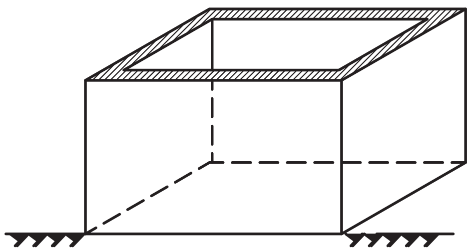
-هى مجموعة من الحوائط و الارضيات الخرسانية المسلحة تحوى داخلها حجم معين من الماء و تنقسم الـ(tanks) الى

1– According to the position of tank with ground level

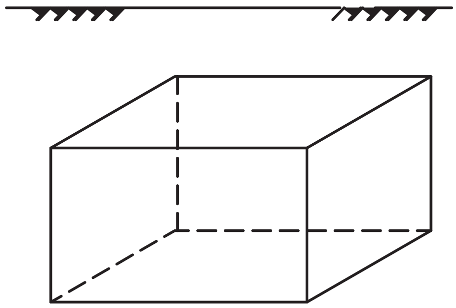
a– Elevated



b– Rested

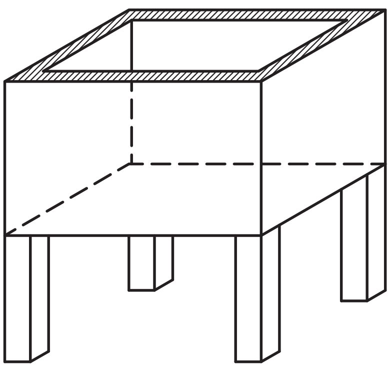


c– Under ground

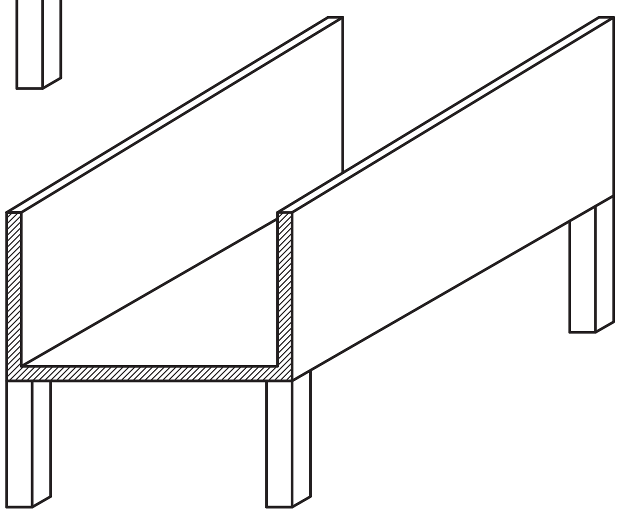


2– According to the shape of tank

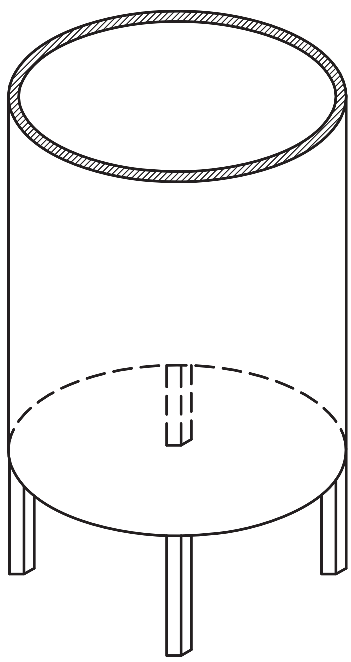
a– Rectangular



b– Open channel



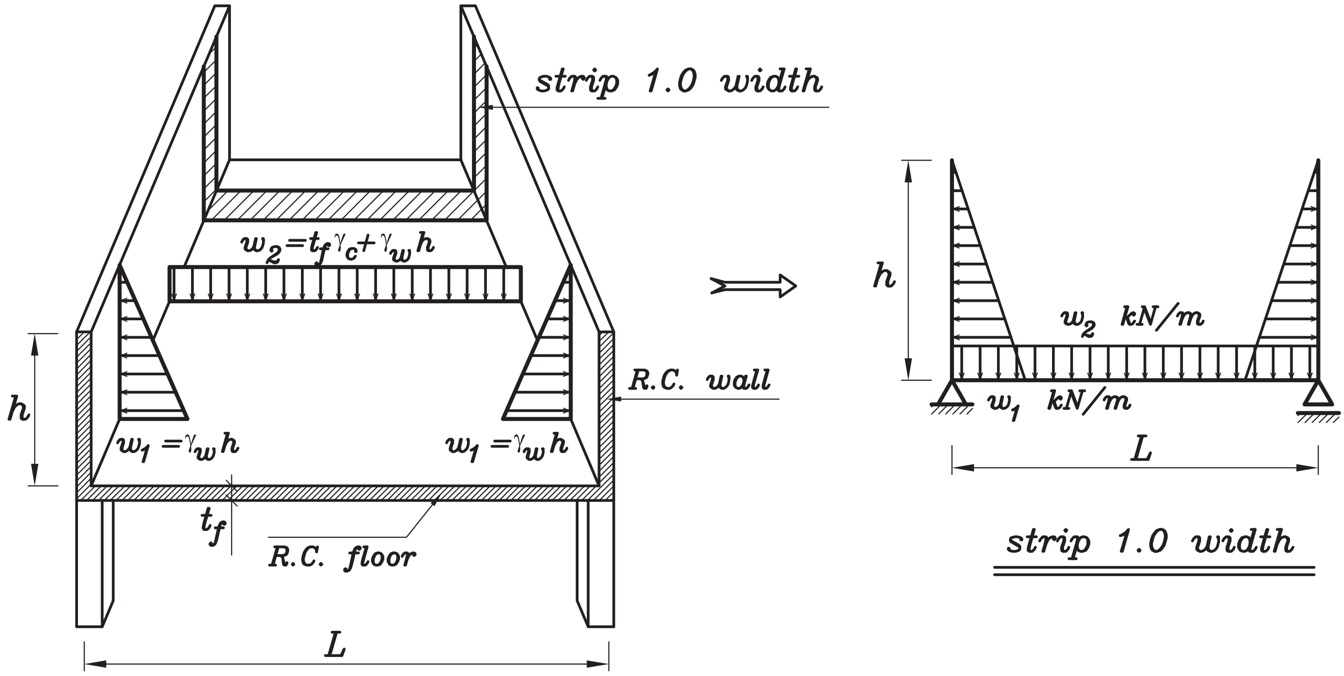
c– Circular



– Statical action of water tanks

-للتعامل مع الخزانات نعتبر حوائط الخزان عبارة عن بلاطات راسية و ارضية الخزان عبارة عن بلاطة افقية ثم ناخذ شريحة بعرض (١ م) و ندرس الاحمال الواقعة عليها .

a– Open channel



where

$$\boxed{w_1 = \gamma_w h} \quad kN/m^2$$

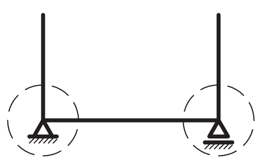
حيث (w_1) هو قيمة ضغط الماء (water pressure)

$$\boxed{w_2 = t_f \gamma_c + \gamma_w h} \quad kN/m^2$$

حيث (w_2) هو وزن وحدة المساحات من الارضية بالاضافة الى وزن عمود الماء الواقع فوقها

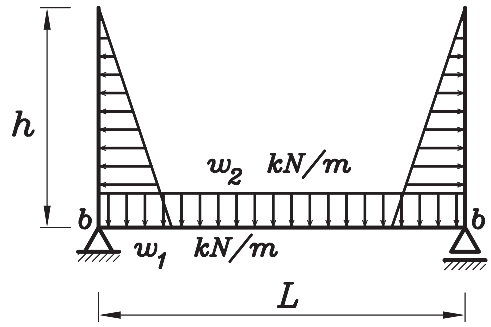
$$\boxed{\gamma_c = 25 \ kN/m^3 \qquad \& \qquad \gamma_w = 10 \ kN/m^3}$$

ـ ملحوظة هامة جدا

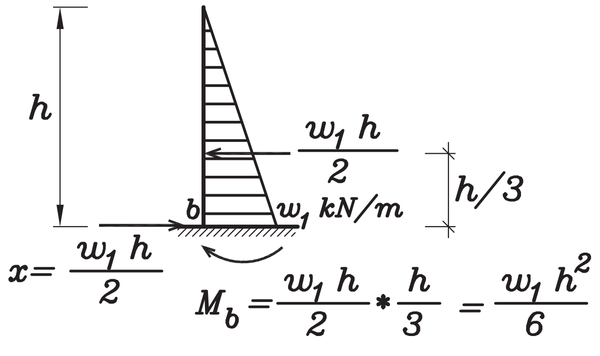


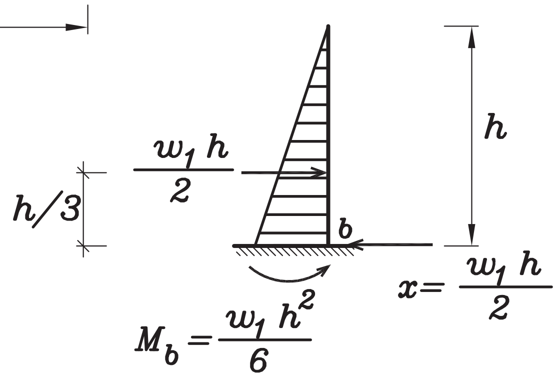
هذا الـ (support) لا يمثل العمود و انما يمثل ان الحائط يعمل ككمرة مقلوبة بالنسبة للارضية .

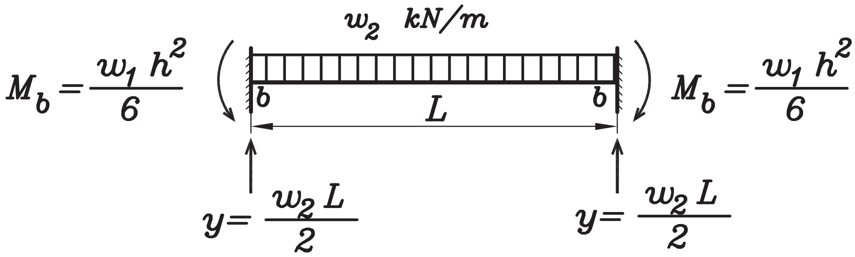
– Analysis of strip:



strip 1.0 width





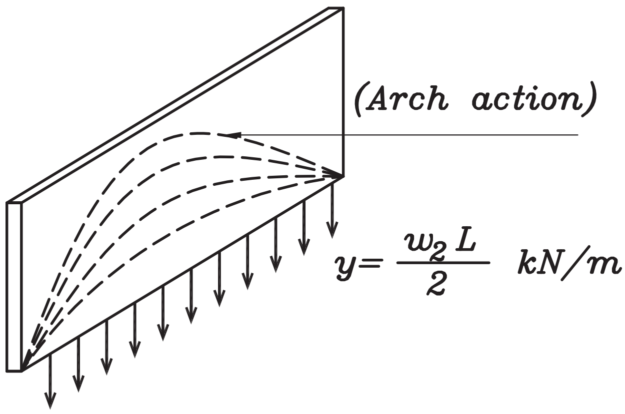


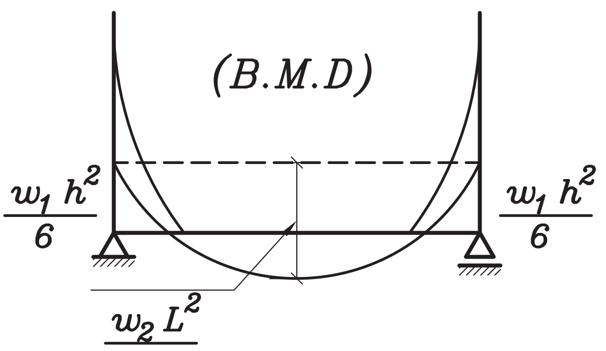
- العزم $[M_b=\frac{w_1 h^2}{6}]$ ينتقل من الحائط الى الارضية لان [joint (b) is a rigid joint]

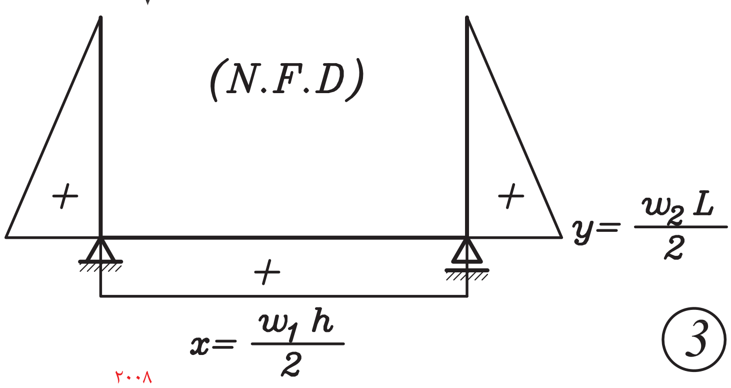
- رد الفعل $[x=\frac{w_1 h}{2}]$ ينتقل من الحائط الى الارضية مسببا شد على الارضية

- رد الفعل $[y=\frac{w_2 L}{2}]$ ينتقل من الارضية الى الحائط مسببا شد على الحائط

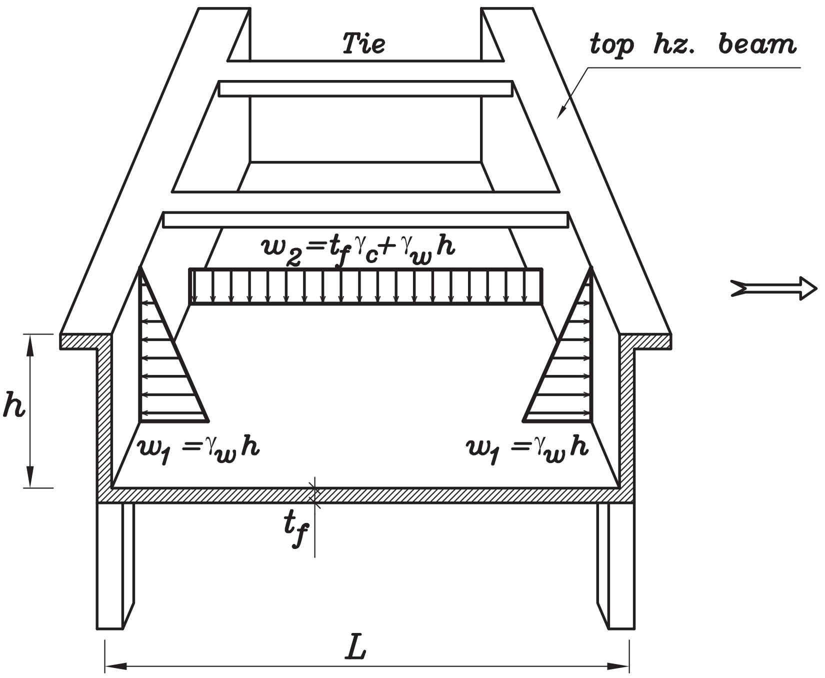
وهذا الشد يقل تدريجيا الى ان يساوى صفر عند اعلى الحائط نتيجة (Arch action)

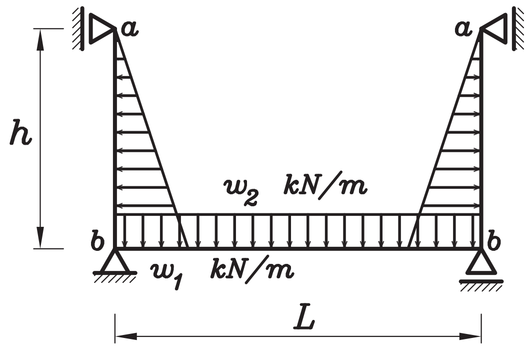






b– Open channel with top horizontal beam





strip 1.0 width

فى هذا ال (tank) نضع كمرة افقية عند اعلى كل حائط وتتزن الكمرتان مع بعضهما بواسطة (tie) يربط بينهما

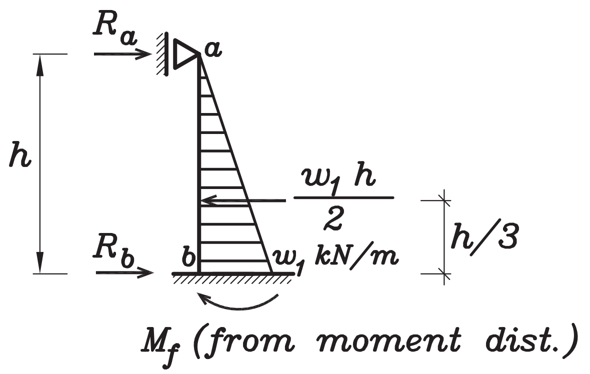
where

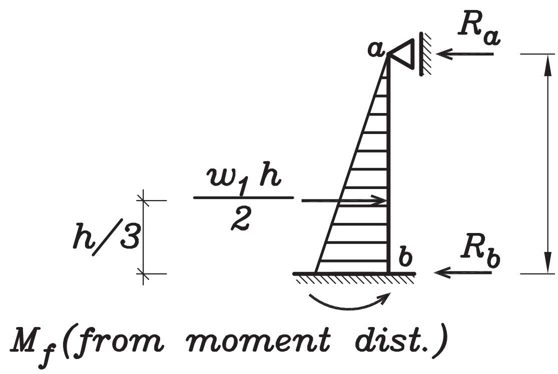
$$w_1 = \gamma_w h \quad \& \quad w_2 = t_f \gamma_c + \gamma_w h \qquad kN/m^2$$

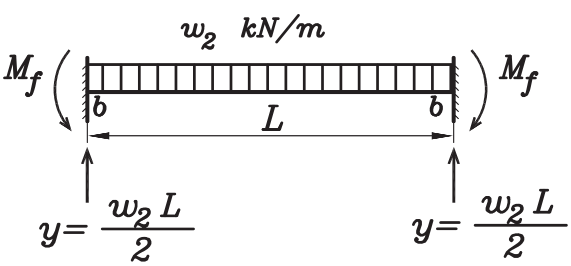
و هذه الشريحة (indeterminate) تحل باستخدام (moment distribution)

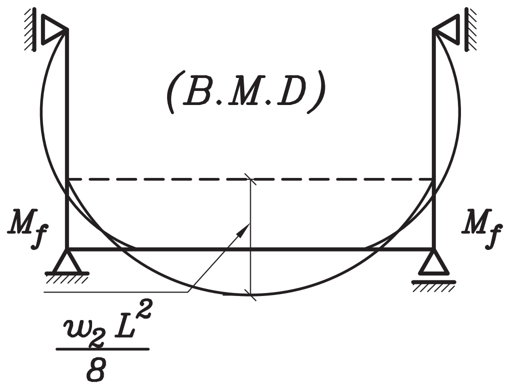
و منها نرسم (B.M.D. & N.F.D.)

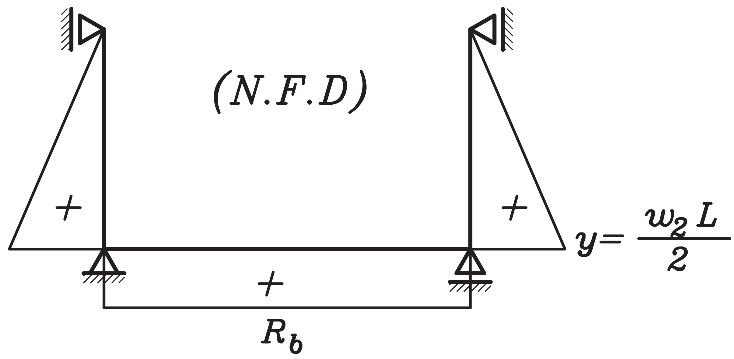
– Analysis of strip:





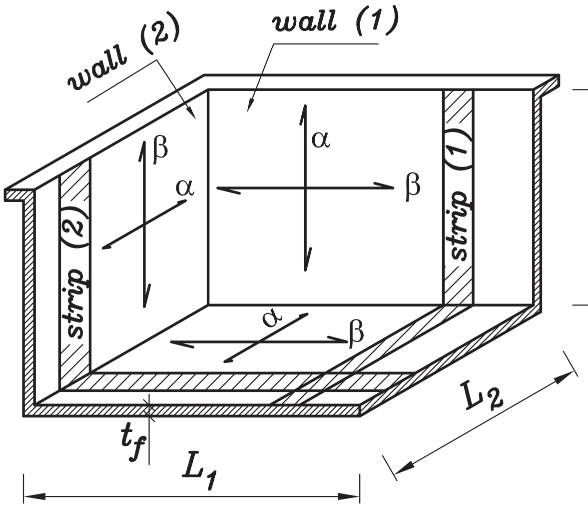


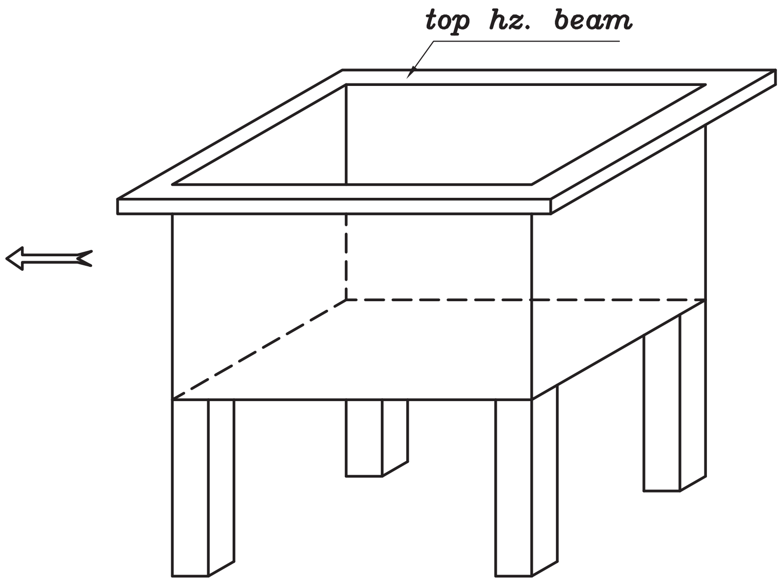




c− Retangular tank

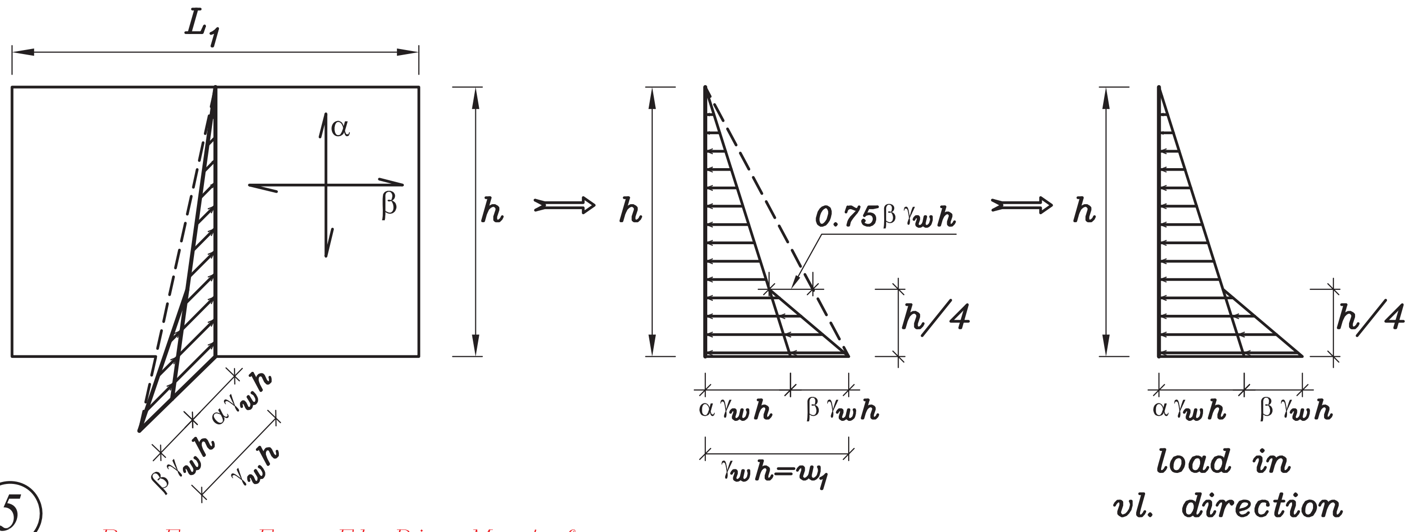
فى حالة (open channel tanks) تكون حوائط الخزان (one way) فى الاتجاه الراسى اما فى حالة (rectangular tank) تكون حوائط الخزان (one or two way) حسب ابعاد الخزان .





$$w_1 = \gamma_w h \quad \& \quad w_2 = t_f \gamma_c + \gamma_w h \qquad kN/m^2$$

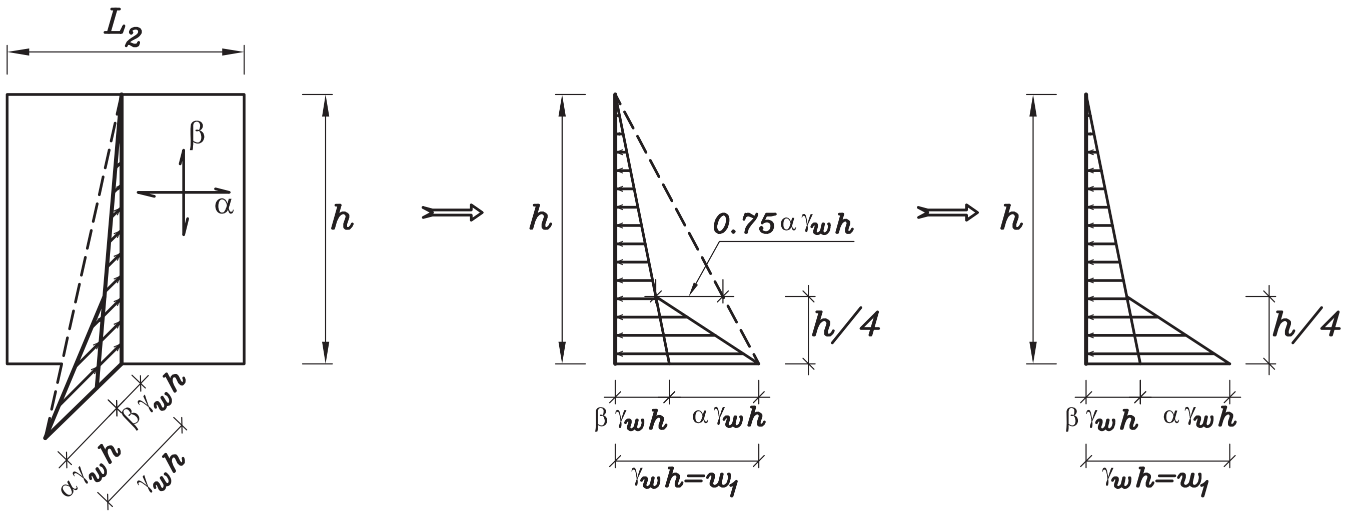
− for wall (1)



نتيجة ان الحائط يعمل كبلاطة راسية *(two way slab)* فان جزء من الحمل يذهب فى الاتجاه الراسى $(\alpha \gamma_w h)$ وباقى الحمل يتجه فى الاتجاه الافقى $(\beta \gamma_w h)$

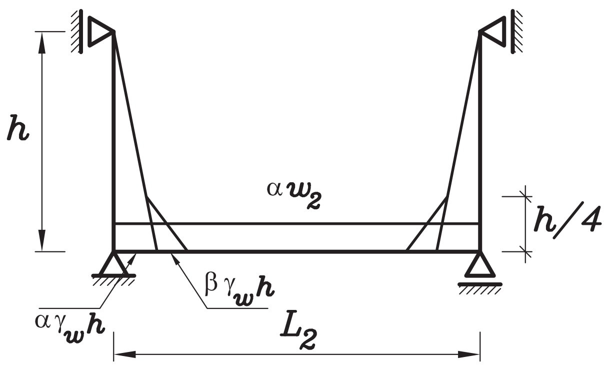
و نتيجة اتصال الحائط بارضية الخزان فان الحائط يكون فى هذه المنطقة *(very rigid)* وبالتالى فان مثلث الماء يذهب كله فى الاتجاه الراسى عند قاعدة الخزان و يقل تدريجيا حتى ارتفاع $(\frac{h}{4})$ كما بالرسم .

– for wall (2)



load in
vl. direction

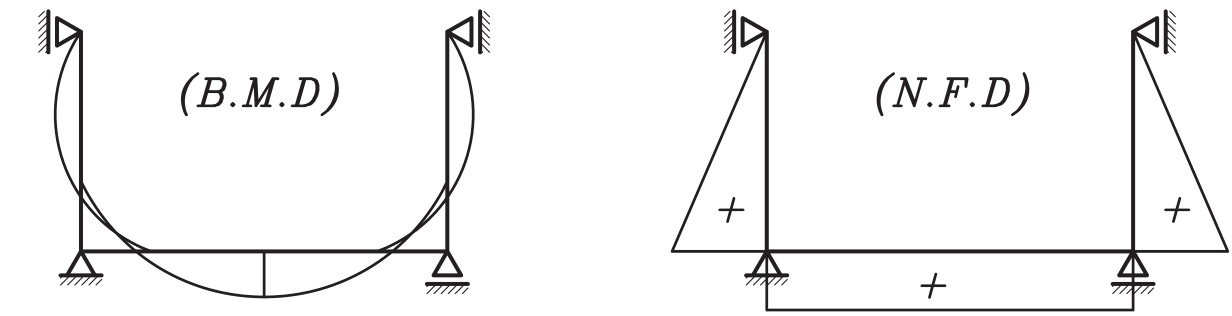
– VL. strip (1)



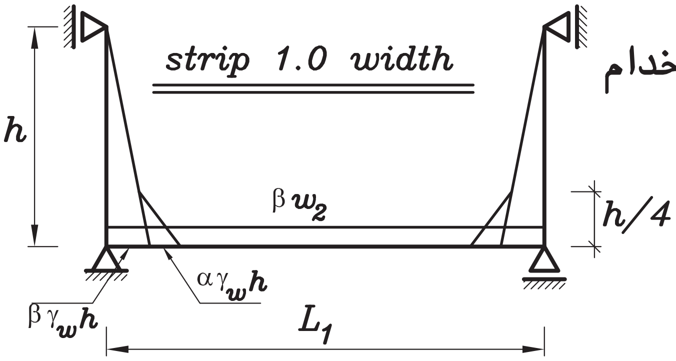
strip 1.0 width

و هذه الشريحة *(indeterminate)* تحل باستخدام *(moment distribution)*

و منها نرسم *(B.M.D. & N.F.D.)*



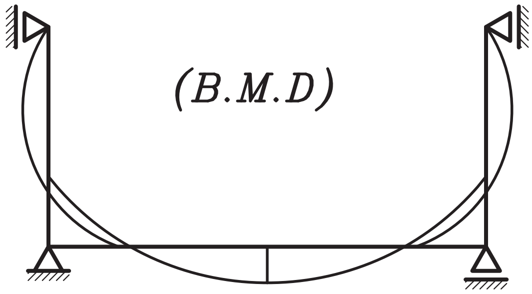
– VL. strip (2)

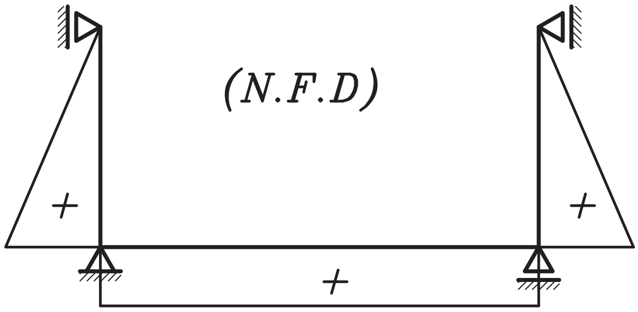


و هذه الشريحة (indeterminate) تحل باستخدام

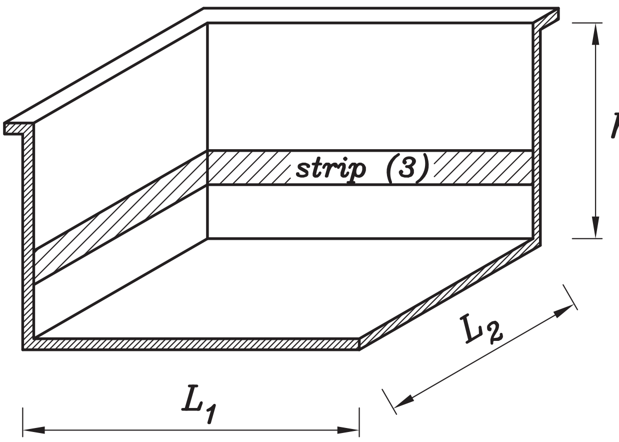
(moment distribution)

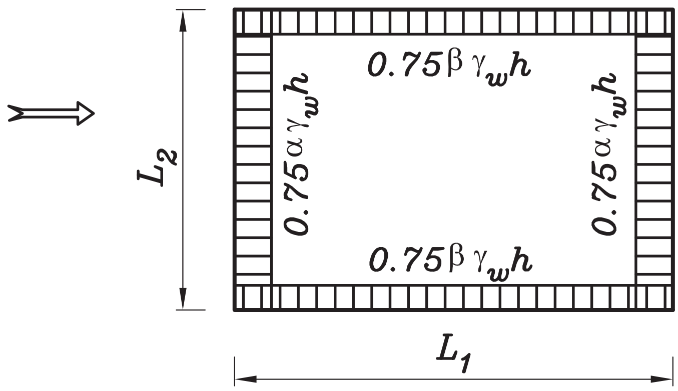
و منها نرسم (B.M.D. & N.F.D.)





– HZ. strip (3) at (h/4) from floor

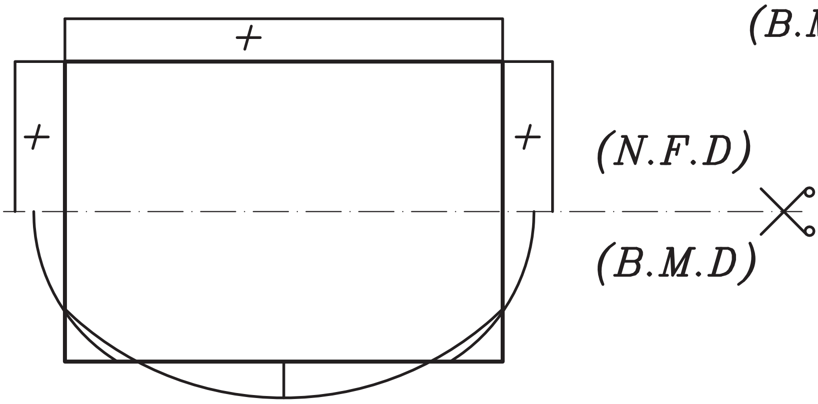




هذه الشريحة الافقية يكون الحمل المؤثر عليها متغير حسب الارتفاع لذلك فان اكبر (hz. load) يؤثر عليها يكون عند ارتفاع (h/4) من قاعدة الخزان

و هذه الشريحة (indeterminate) تحل باستخدام (moment distribution)

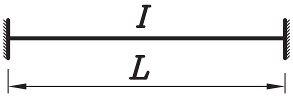
و منها نرسم (B.M.D. & N.F.D.)



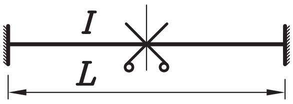
– Revision on moment distribution method

1–Get the relative stiffness for each member

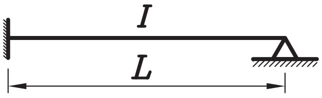
Fixed Fixed $K=\frac{I}{L}$



Symmetrical $K=\frac{0.5I}{L}$



Fixed Hinged $K=\frac{0.75I}{L}$

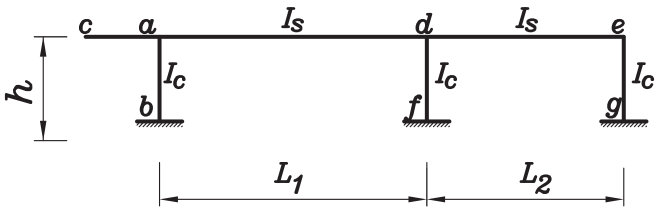


2–Get the distribution factor of each member

$$D.f = \frac{K_{member}}{\sum K_{member}}$$

$$D.f_{ab} = \frac{(I_c/h)}{(I_s/L_1)+(I_c/h)}$$

$$D.f_{ad} = \frac{(I_s/L_1)}{(I_s/L_1)+(I_c/h)}$$

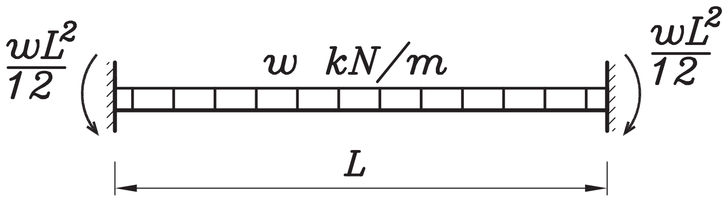


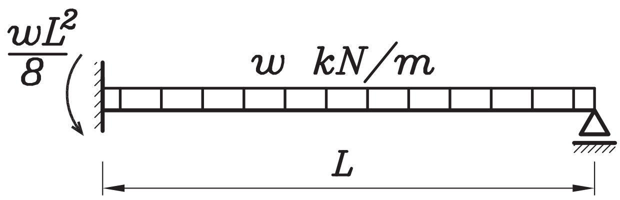
Note: $D.f_{ac} =$ zero (cantilever)

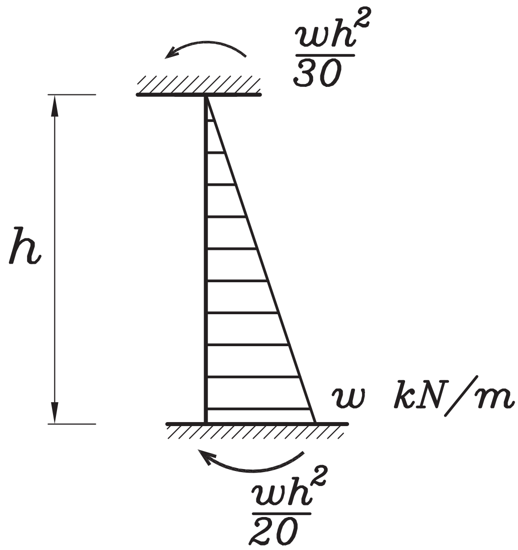
$D.f_{ba} =$ zero (fixation)

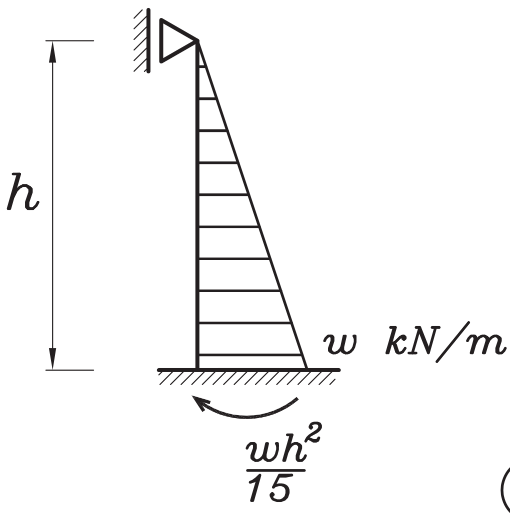
و بالمثل لباقى ال (joints)

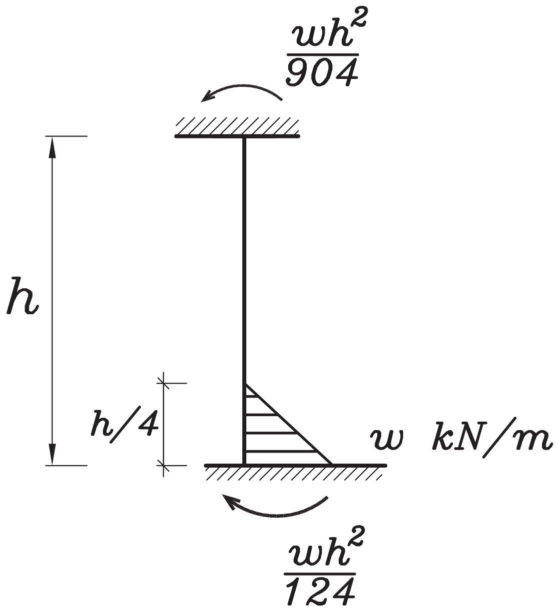
3–Get the F.E.M of each member

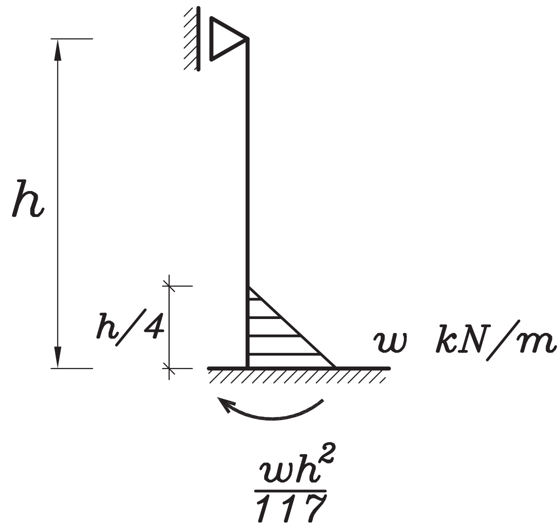












Sign convention اصطلاح الاشارات

العزوم التى تلف مع عقارب الساعة *(clockwise)* تكون موجبة

و التى تلف ضد عقارب الساعة *(anticlockwise)* تكون سالبة

4-Make moment distribution table

بعمل جدول *(moment distribution)* يمكن حساب العزوم عند كل *(joint)*

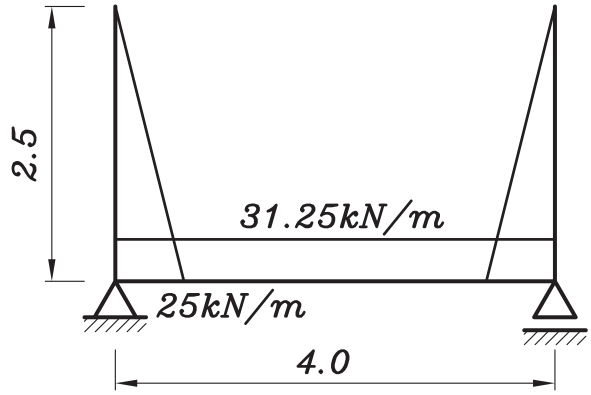
ومنه يمكن رسم *(B.M.D.)*

Joint	b	a			d			f	. .
member	ba	ab	ac	ad	da	de	df	fd	
D.f.	0	✓	0	✓	✓	✓	✓	0	
F.E.M.	0	0	✓	✓	✓	0	✓	0	
Bal.M.	0	✓	0	✓	✓	✓	✓	0	
C.O.M.	✓	0	0	✓	✓	✓	0	✓	
Bal.M.	0	✓	✓	✓	✓	✓	✓	0	
M_f	✓	✓	✓	✓	✓	✓	✓	✓	

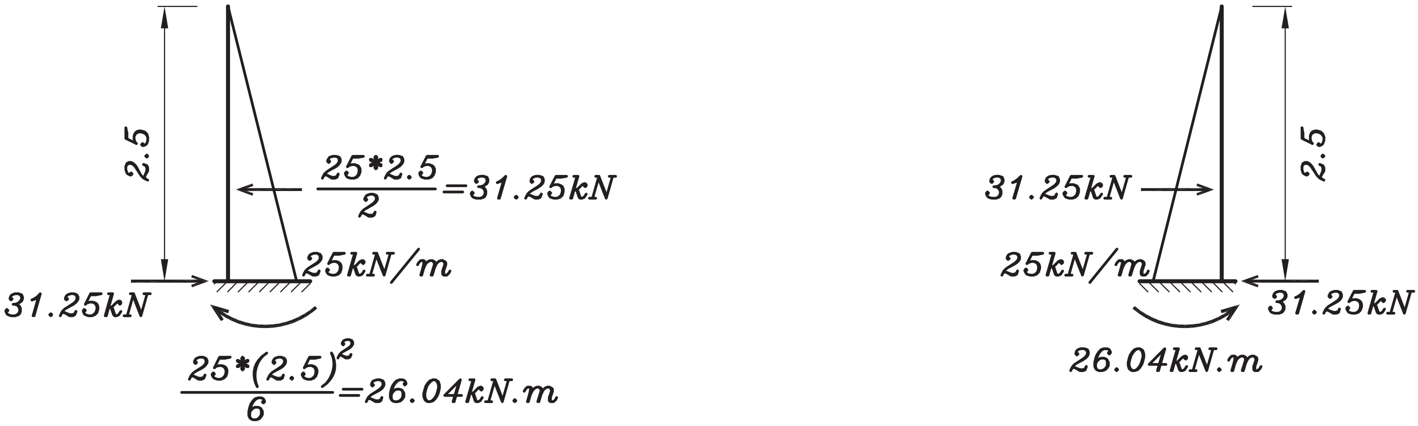
*0.5   *0.5

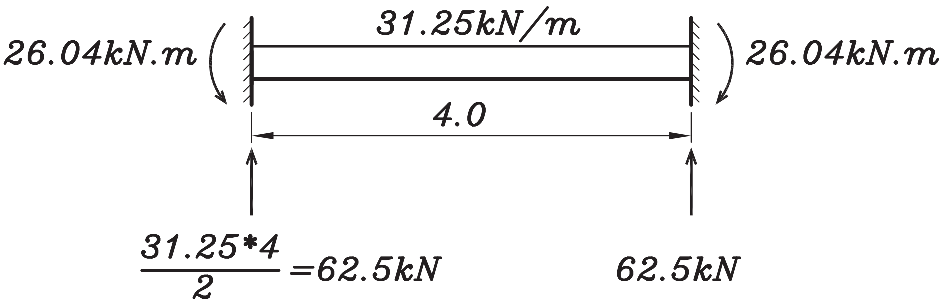
– For the following figures, it is required to draw B.M.D. & N.F.D.

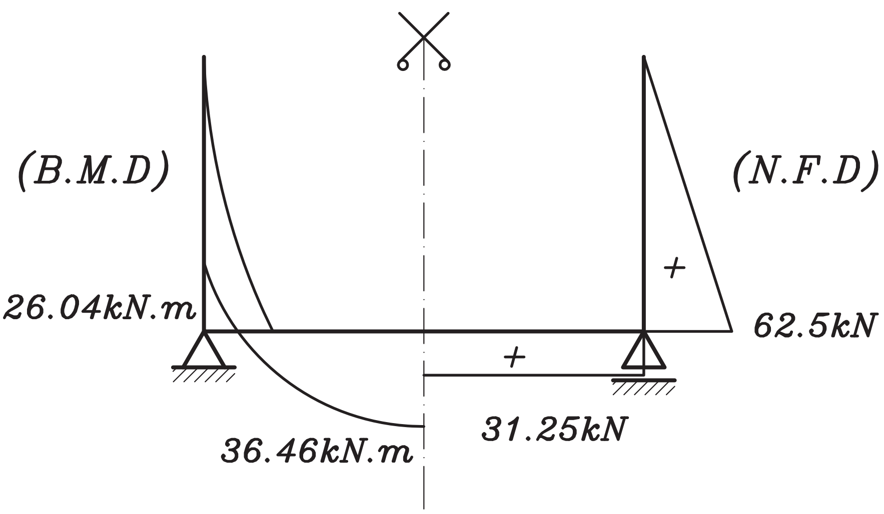
Example(1)



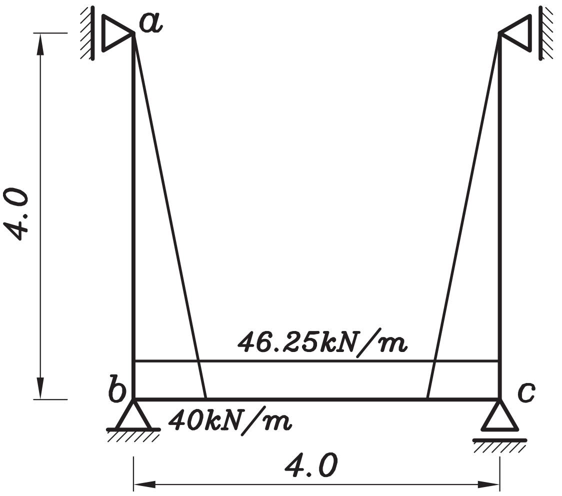
strip 1.0 width







Example(2)

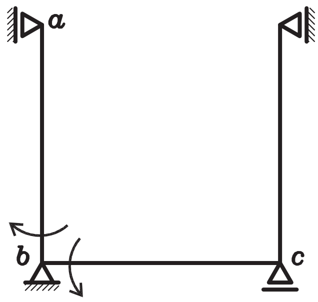


strip 1.0 width

For Joint b

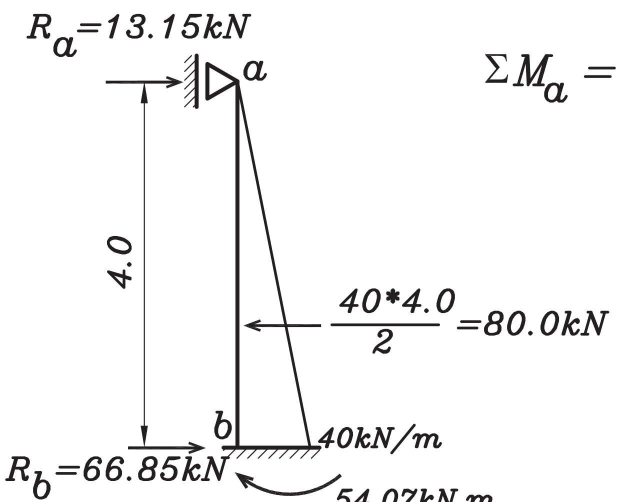
$$D.f_{ba} = \frac{0.75(I/4.0)}{0.75(I/4.0)+0.5(I/4)} = 0.6$$

$$D.f_{bc} = \frac{0.5(I/4)}{0.75(I/4.0)+0.5(I/4)} = 0.4$$



$$F.E.M._{ba} = \frac{40*(4)^2}{15} = 42.67kN.m \quad , \quad F.E.M._{bc} = \frac{-46.25*(4)^2}{12} = -61.67kN.m$$

Joint	b	
member	ba	bc
D.f.	0.60	0.40
F.E.M.	42.67	−61.67
Bal.M.	11.40	7.60
M_f	54.07	−54.07



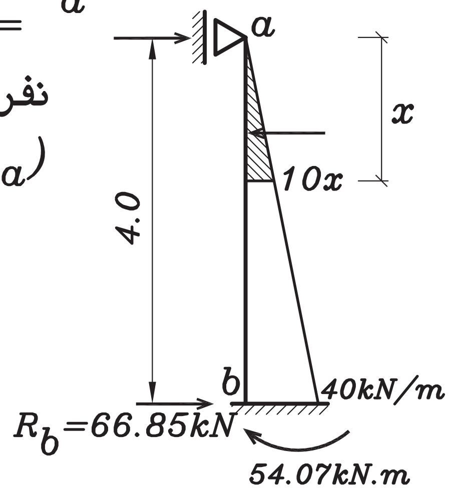
$$\Sigma M_a = 0 \implies 80*(\tfrac{2}{3}*4)+54.07-R_b*4=0$$

$$\implies R_b = 66.85kN$$

$$\Sigma x = 0 \implies R_a + R_b = 80$$

$$\implies R_a = 13.15kN$$

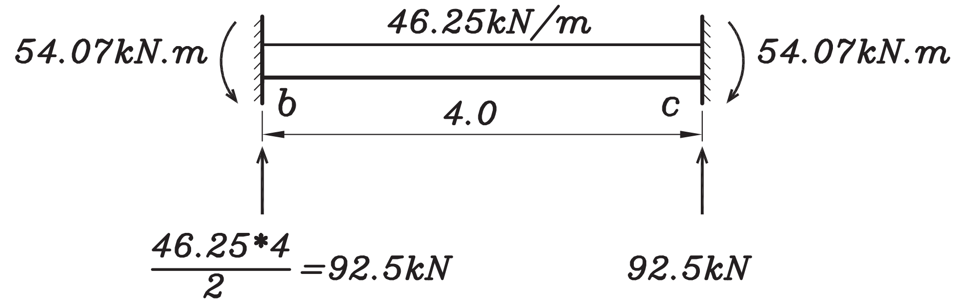
To get max M_{+ve} at point of zero shear

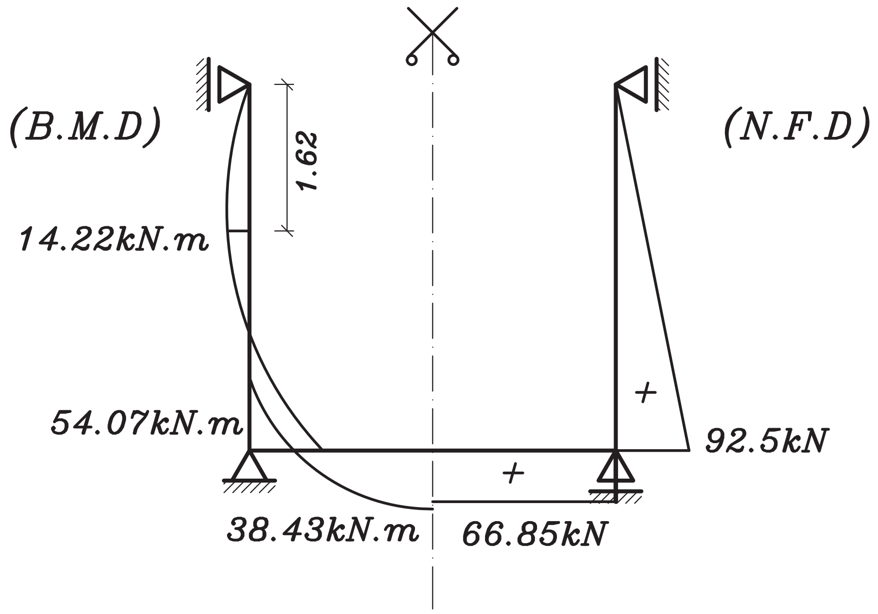


نفرض ان (zero shear) على ارتفاع (x) من اعلى و بمساواة (R_a) بمساحة المثلث المهشر نحصل على قيمة (x)

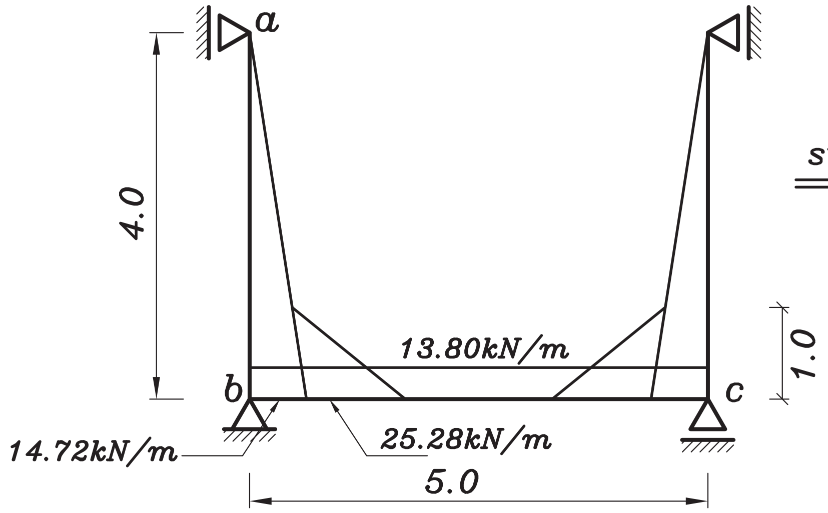
$$10x\left(\frac{x}{2}\right)=R_a=13.15 \Longrightarrow x=1.62m$$

$$\Longrightarrow M_{+ve}=13.15x-10x\left(\frac{x^2}{6}\right)=14.22\ \ kN.m$$



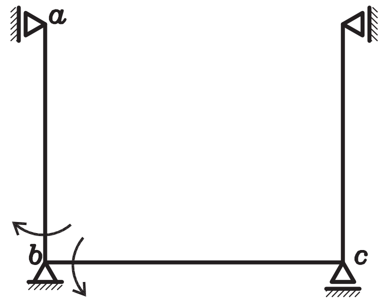


Example(3)



strip 1.0 width

For Joint b



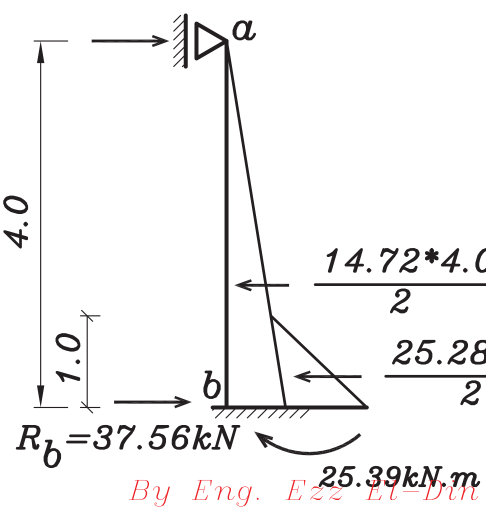
$$D.f_{ba} = \frac{0.75(I/4.0)}{0.75(I/4.0)+0.5(I/5)} = 0.65$$

$$D.f_{bc} = \frac{0.5(I/5)}{0.75(I/4.0)+0.5(I/5)} = 0.35$$

$$F.E.M._{ba} = \frac{14.72*(4)^2}{15} + \frac{25.28*(4)^2}{117} = 19.16kN.m$$

$$F.E.M._{bc} = \frac{-13.80*(5)^2}{12} = -28.75kN.m$$

Joint	b	
member	ba	bc
D.f.	0.65	0.35
F.E.M.	19.16	−28.75
Bal.M.	6.23	3.36
M_f	25.39	−25.39

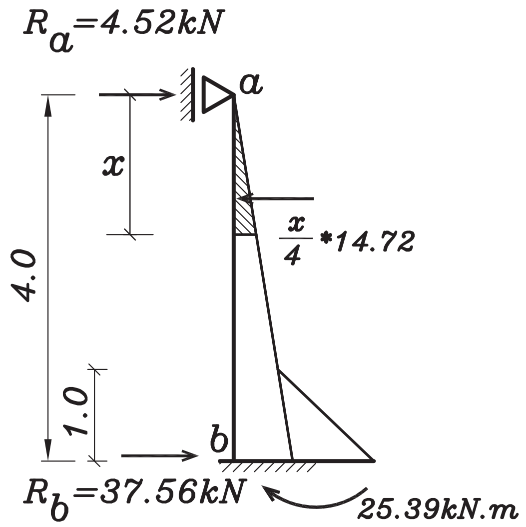


$$\frac{14.72*4.0}{2} = 29.44kN$$

$$\frac{25.28*1.0}{2} = 12.64kN$$

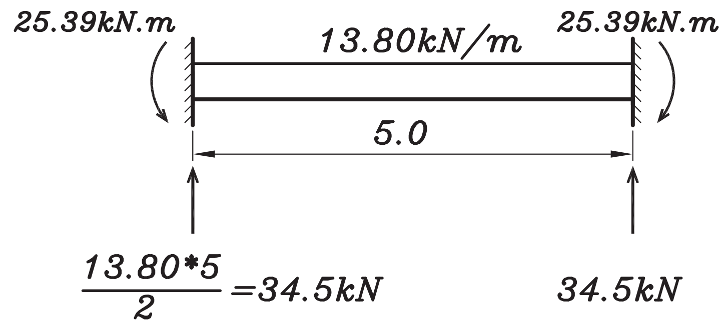
$$\Sigma M_a = 0 \Longrightarrow 29.44*(\tfrac{2}{3}*4)+12.64*(3+\tfrac{2}{3}*1)$$
$$+25.39-R_b*4=0$$
$$\Longrightarrow R_b=37.56kN$$
$$\Sigma x = 0 \Longrightarrow R_a+R_b=29.44+12.64$$
$$\Longrightarrow R_a=4.52kN$$

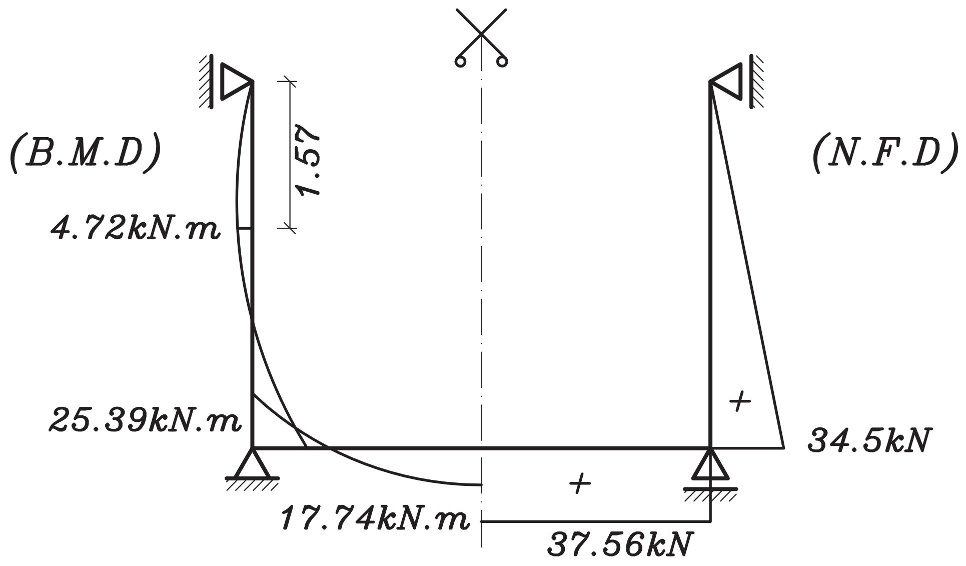
To get max M_{+ve} at point of zero shear



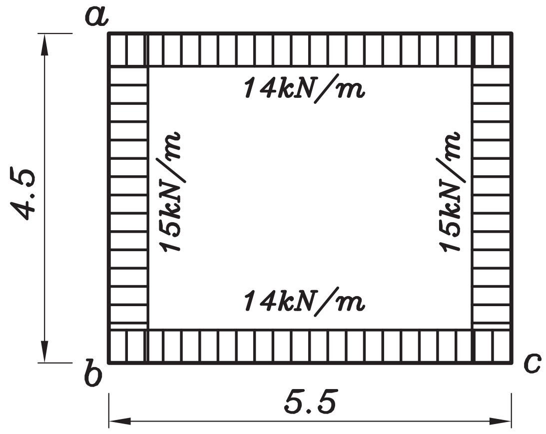
$$\left[\frac{x}{4}*14.72\right]*\frac{x}{2}=4.52 \Longrightarrow x=1.57m$$

$$\Longrightarrow M_{+ve}=4.52x-\left(\frac{x}{4}*14.72\right)*\left(\frac{x^2}{6}\right)=4.72\ \ kN.m$$

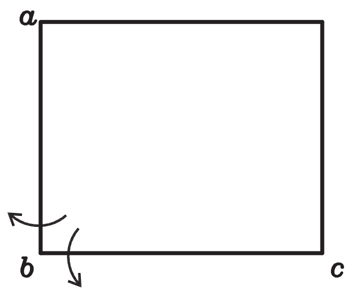




Example(4)



strip 1.0 width



For Joint b

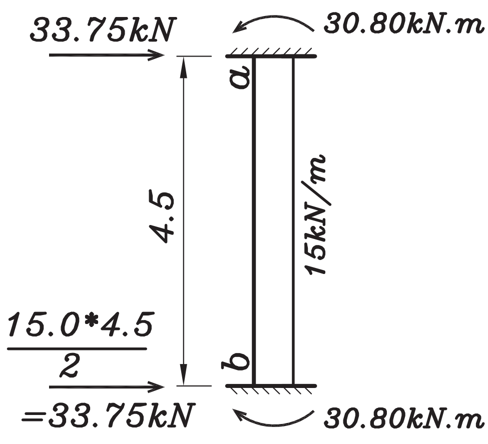
$$D.f_{ba} = \frac{0.5(I/4.5)}{0.5(I/4.5)+0.5(I/5.5)} = 0.55$$

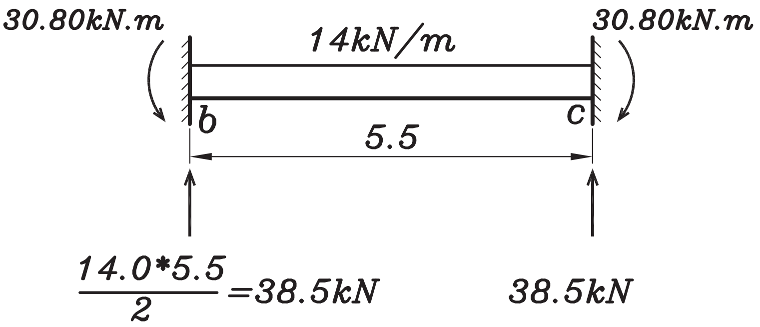
$$D.f_{bc} = \frac{0.5(I/5.5)}{0.5(I/4.5)+0.5(I/5.5)} = 0.45$$

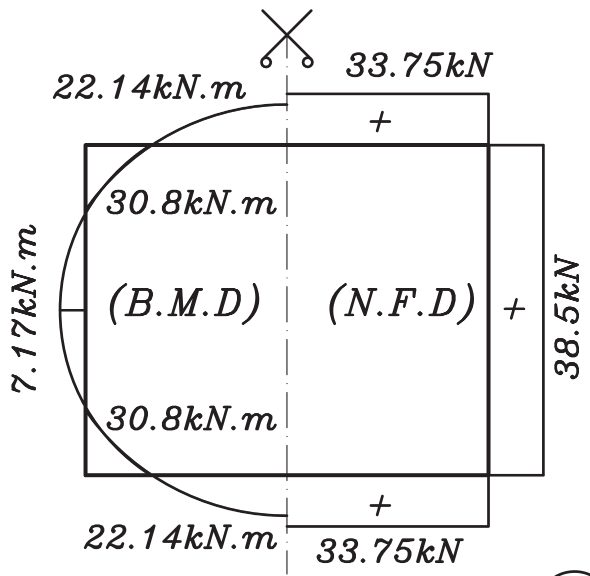
$$F.E.M._{ba} = \frac{15.0*(4.5)^2}{12} = 25.31 kN.m$$

$$F.E.M._{bc} = \frac{-14.0*(5.5)^2}{12} = -35.29 kN.m$$

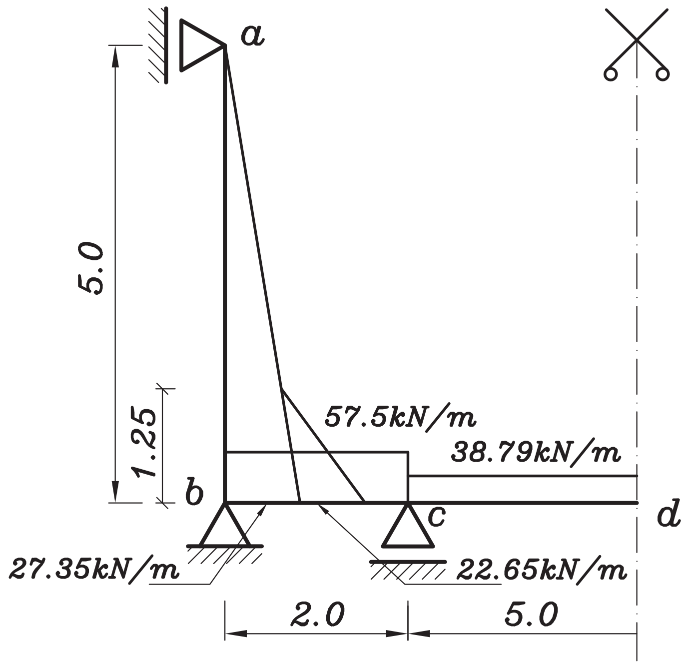
Joint	b	
member	ba	bc
D.f.	0.55	0.45
F.E.M.	25.31	−35.29
Bal.M.	5.49	4.49
M_f	30.80	−30.80







Example(5)

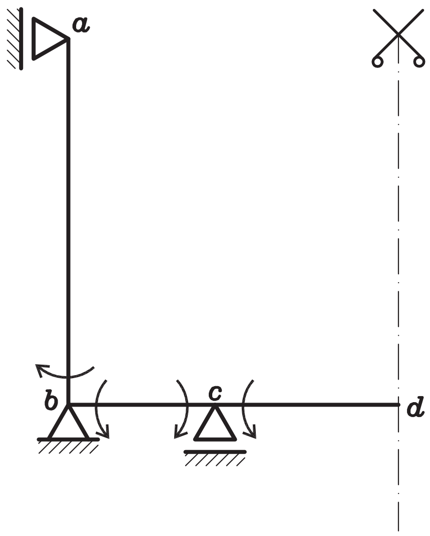


strip 1.0 width

For Joint b

$$D.f_{ba} = \frac{0.75(I/5.0)}{0.75(I/5.0)+(I/2.0)} = 0.23$$

$$D.f_{bc} = \frac{(I/2.0)}{0.75(I/5.0)+(I/2.0)} = 0.77$$



For Joint c

$$D.f_{cb} = \frac{(I/2.0)}{(I/2.0)+0.5(I/5.0)} = 0.83$$

$$D.f_{cd} = \frac{0.5(I/5.0)}{(I/2.0)+0.5(I/5.0)} = 0.17$$

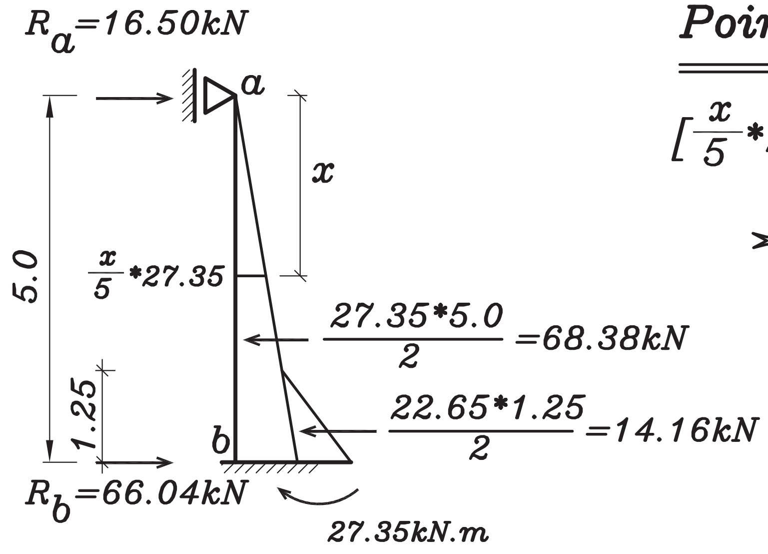
$$F.E.M._{ba} = \frac{27.35*(5)^2}{15} + \frac{22.65*(5)^2}{117} = 50.42kN.m$$

$$F.E.M._{bc} = \frac{-57.5*(2.0)^2}{12} = -19.17kN.m \quad , \quad F.E.M._{cb} = 19.17kN.m$$

$$F.E.M._{cd} = \frac{-38.79*(5.0)^2}{12}$$

$$F.E.M._{cd} = -80.81kN.m$$

Joint	b		c	
member	ba	bc	cb	cd
D.f.	0.23	0.77	0.83	0.17
F.E.M.	50.42	−19.17	19.17	−80.81
Bal.M.	−7.19	−24.06	51.16	10.48
C.O.M.	0	25.58	−12.03	0
Bal.M.	−5.88	−19.70	9.98	2.05
M_f	37.35	−37.35	68.28	−68.28

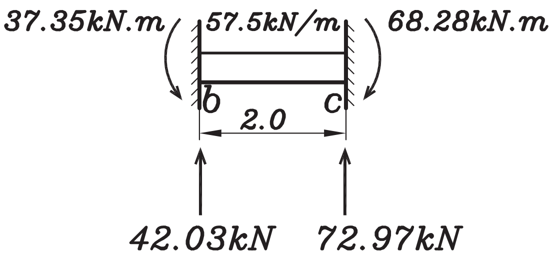


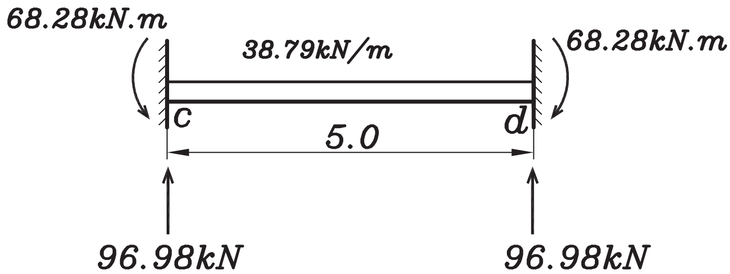
Point of zero shear

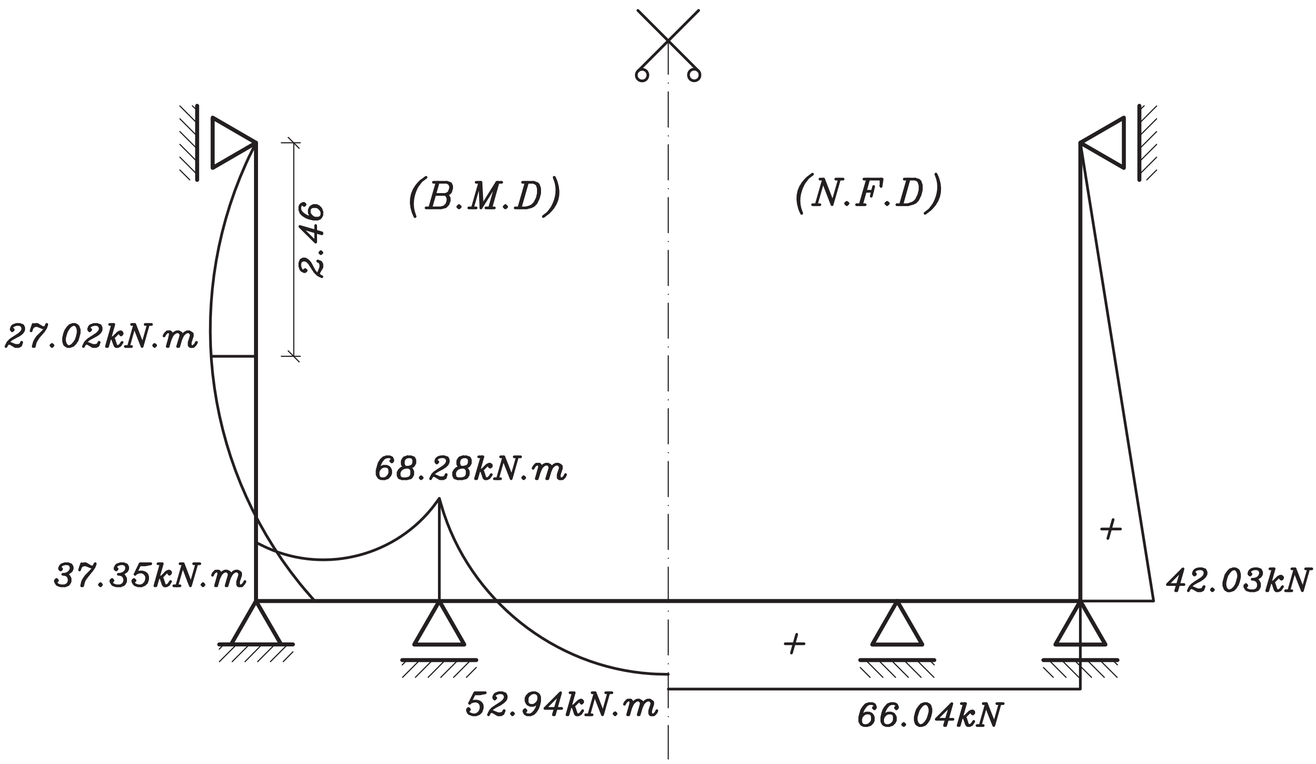
$$[\frac{x}{5}*27.35]*\frac{x}{2}=16.5 \Longrightarrow x=2.46m$$

$$\Longrightarrow M_{+ve}=16.5x-(\frac{x}{5}*27.35)*(\frac{x^2}{6})$$

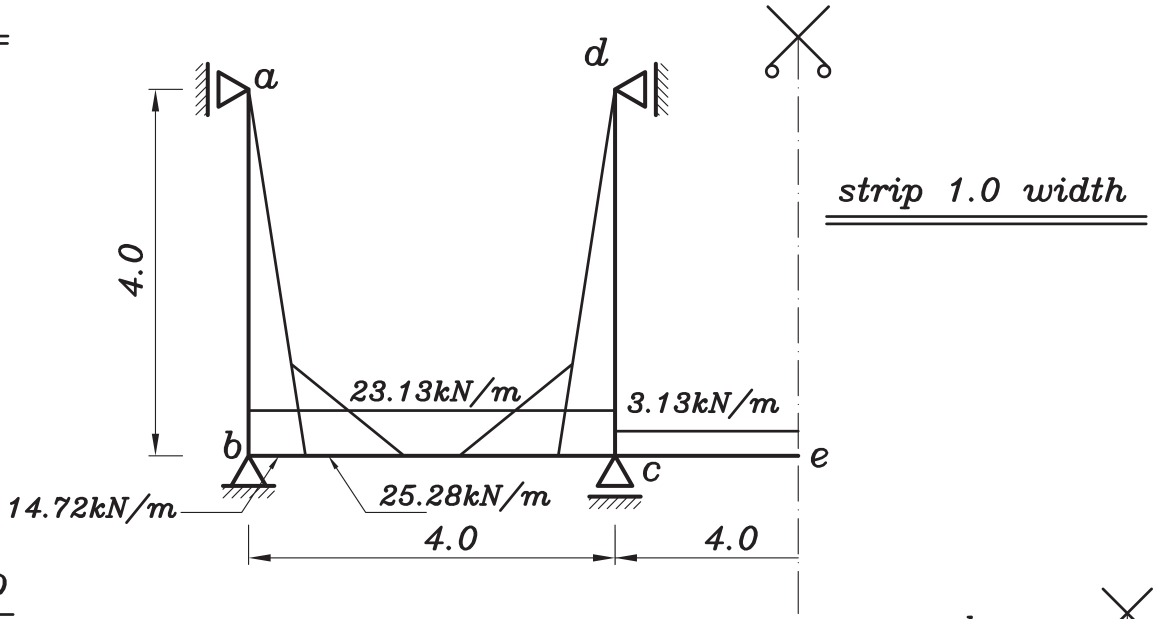
$$M_{+ve}=27.02 \ \ kN.m$$



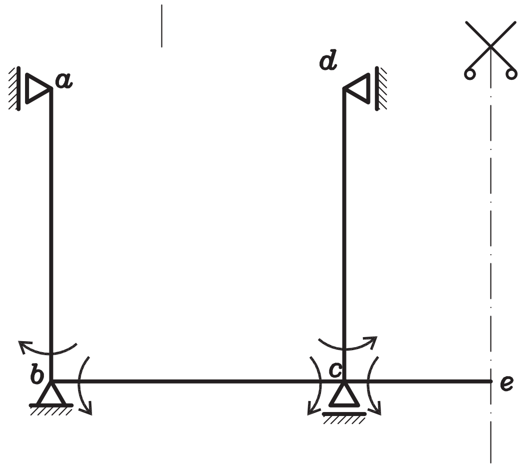




Example(6)



For Joint b



$$D.f_{ba} = \frac{0.75(I/4.0)}{0.75(I/4.0)+(I/4.0)} = 0.43$$

$$D.f_{bc} = \frac{(I/4.0)}{0.75(I/4.0)+(I/4.0)} = 0.57$$

For Joint c

$$D.f_{cb} = \frac{(I/4.0)}{(I/4.0)+0.75(I/4.0)+0.5(I/4)} = 0.44$$

$$D.f_{cd} = \frac{0.75(I/4.0)}{(I/4.0)+0.75(I/4.0)+0.5(I/4)} = 0.33$$

$$D.f_{ce} = \frac{0.5(I/4.0)}{(I/4.0)+0.75(I/4.0)+0.5(I/4)} = 0.23$$

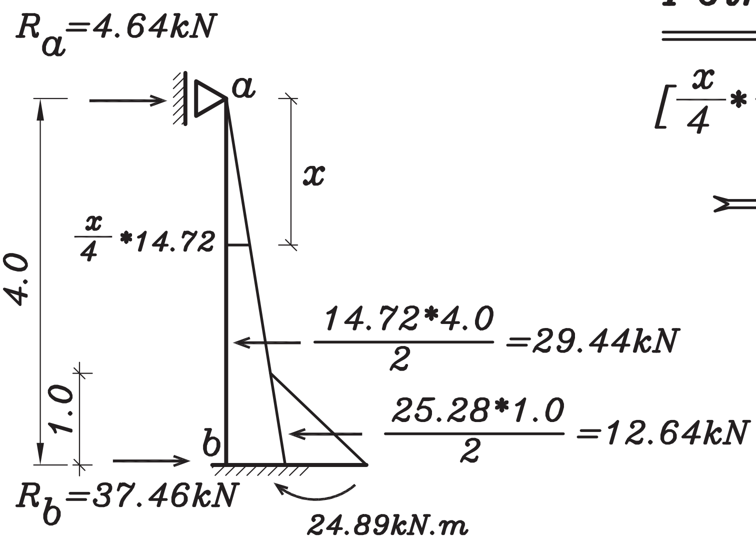
$$F.E.M._{ba} = \frac{14.72*(4)^2}{15} + \frac{25.28*(4)^2}{117} = 19.16kN.m$$

$$F.E.M._{cd} = -19.16kN.m$$

$$F.E.M._{bc} = \frac{-23.13*(4.0)^2}{12} = -30.84kN.m \quad , \quad F.E.M._{cb} = 30.84kN.m$$

$$F.E.M._{ce} = \frac{-3.13*(4.0)^2}{12} = -4.17kN.m$$

Joint	b		c		
member	ba	bc	cb	cd	ce
D.f.	0.43	0.57	0.44	0.33	0.23
F.E.M.	19.16	−30.84	30.84	−19.16	−4.17
Bal.M.	5.02	6.66	−3.30	−2.48	−1.73
C.O.M.	0	−1.65	3.33	0	0
Bal.M.	0.71	0.94	−1.47	−1.10	−0.77
M_f	24.89	−24.89	29.40	−22.74	−6.67



Point of zero shear

$$\left[\frac{x}{4}*14.72\right]*\frac{x}{2}=4.64 \Longrightarrow x=1.59m$$

$$\Longrightarrow M_{+ve}=4.64x-\left(\frac{x}{4}*14.72\right)*\left(\frac{x^2}{6}\right)$$

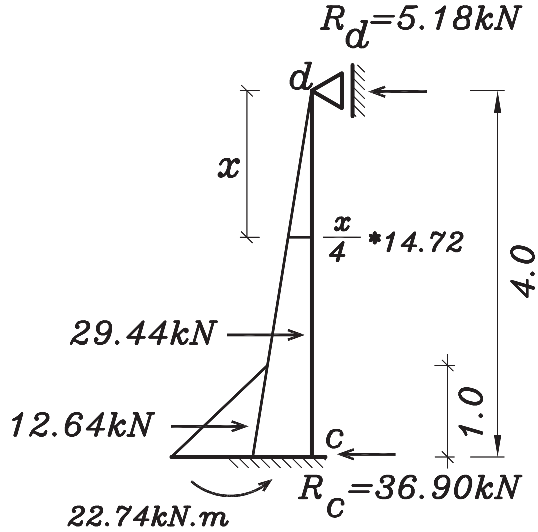
$$M_{+ve}=4.91 \ \ kN.m$$

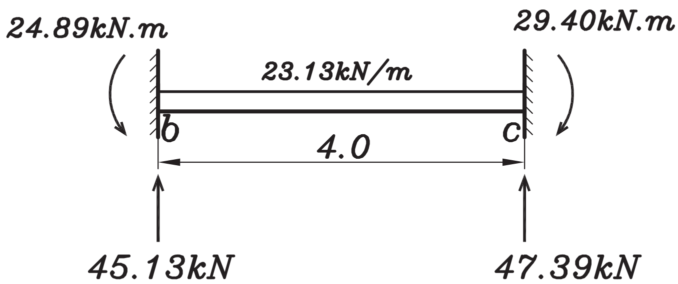
Point of zero shear

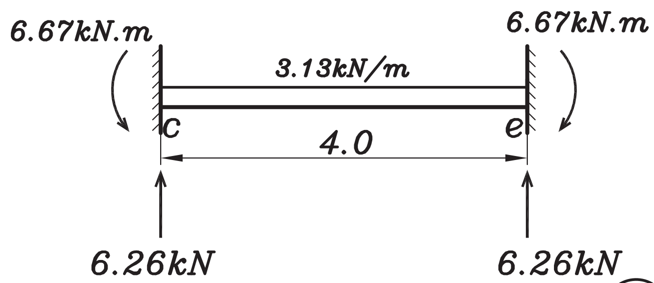
$$\left[\frac{x}{4}*14.72\right]*\frac{x}{2}=5.18 \Longrightarrow x=1.68m$$

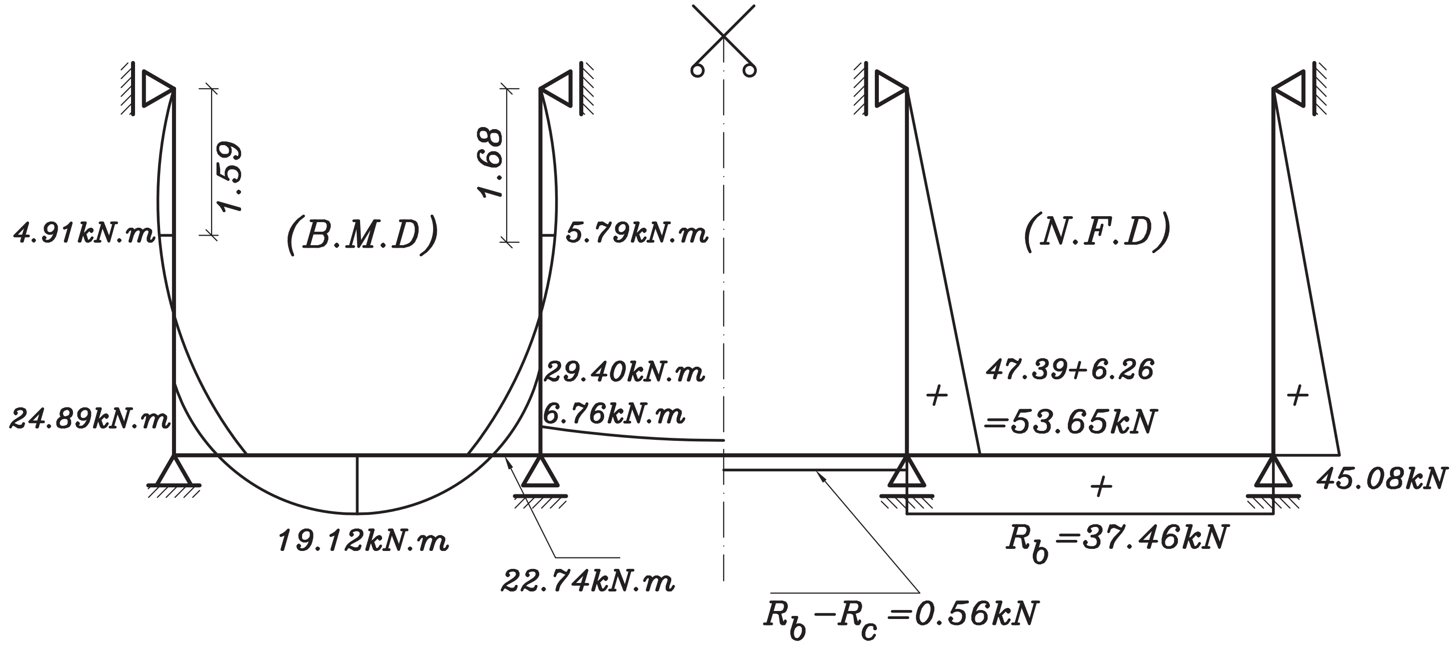
$$\Longrightarrow M_{+ve}=5.18x-\left(\frac{x}{4}*14.72\right)*\left(\frac{x^2}{6}\right)$$

$$M_{+ve}=5.79 \ \ kN.m$$

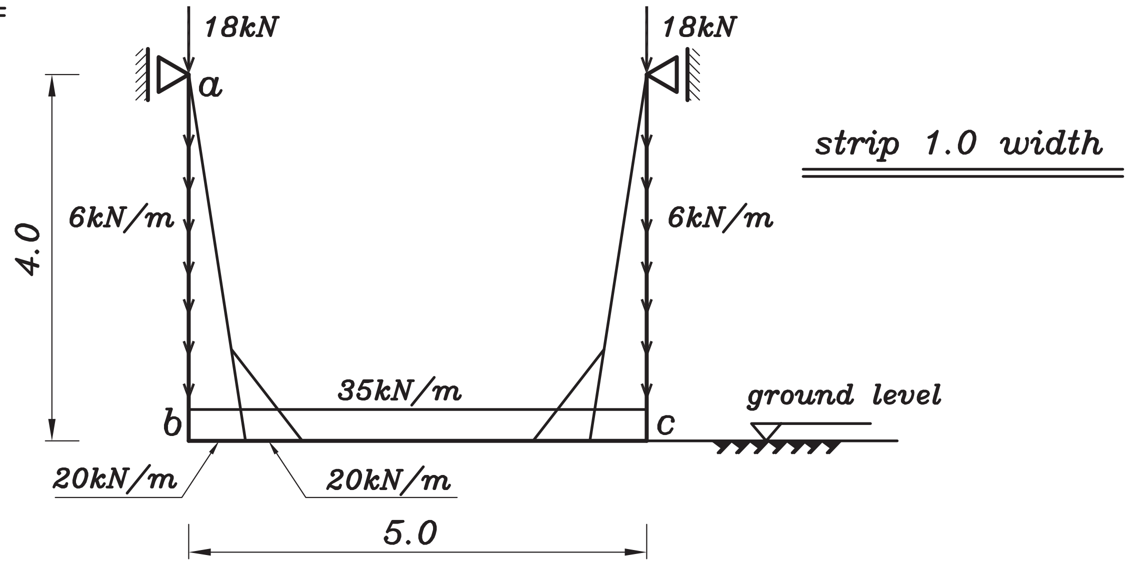








Example(7)



- هذا الخزان يمثل خزان *(rested on soil)* لذلك لكى نضمن ان الاجهادات موزعة على التربة بانتظام *(uniformly distributed)* فان تخانة الارضية (t_f) يجب الا تقل عن $(400-500mm)$ بينما تخانة الحوائط تتراوح بين $(250-300mm)$ لذلك ناخذ $[I_{base} = (4-8) I_{wall}]$
- الحمل الراسى الموزع على الحائط $(6kN/m)$ يمثل $(o.w.)$ للحائط .
- يجب عند التعامل مع الخزان حساب الاجهادات الواقعة على التربة (f_{gross}) و ذلك باعتبار ان ارضية الحزان تعمل ك *(footing)* للخزان باكمله .

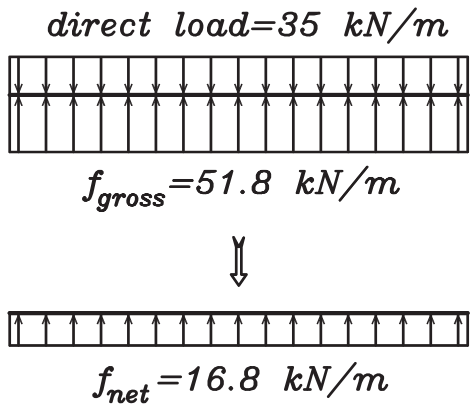
W_{total}= مجموع اوزان الخزان

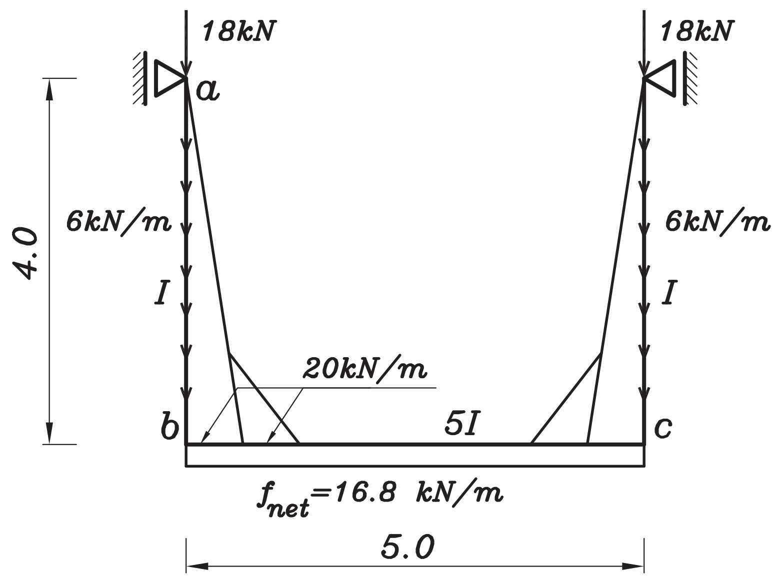
$$W_{total} = \underset{\text{concen. load}}{18*2} + \underset{\text{o.w. of wall}}{6*4*2} + \underset{\text{floor}}{35*5} = 259 \ kN$$

$$f_{gross} = \frac{W_{total}}{A} = \frac{259}{5*1} = 51.8 \ kN/m$$

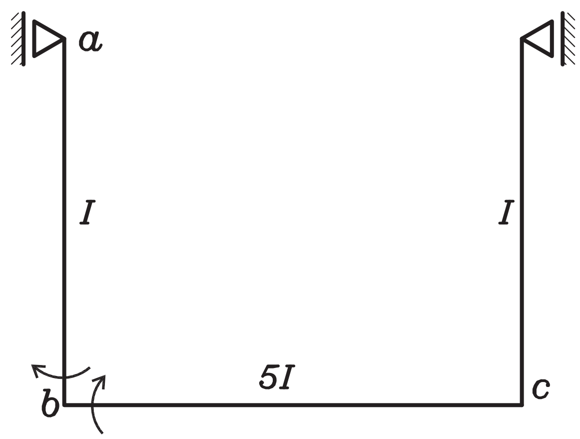
$$f_{net} = f_{gross}\uparrow - \text{direct load}\downarrow$$

$$f_{net} = 51.8\uparrow - 35\downarrow = 16.8\uparrow \ kN/m$$





Assume $I_{base} = 5I_{wall}$



For Joint b

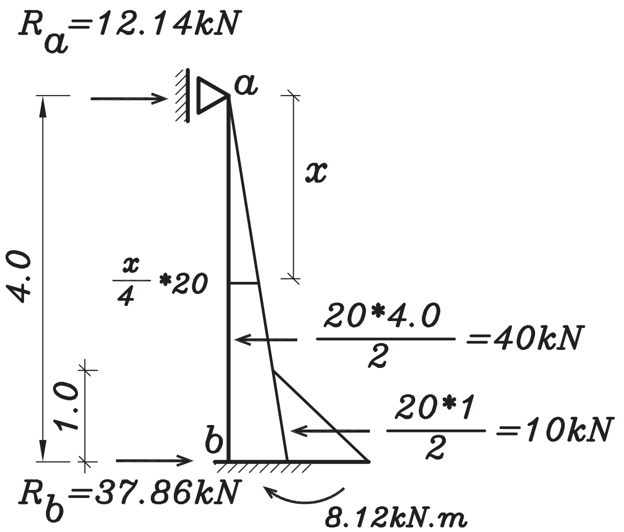
$$D.f_{ba} = \frac{0.75(I/4.0)}{0.75(I/4.0)+0.5(5I/5)} = 0.27$$

$$D.f_{bc} = \frac{0.5(5I/5)}{0.75(I/4.0)+0.5(5I/5)} = 0.73$$

$$F.E.M._{ba} = \frac{20.0*(4)^2}{15} + \frac{20.0*(4)^2}{117} = 24.07 kN.m$$

$$F.E.M._{bc} = \frac{16.8*(5.0)^2}{12} = 35.0 kN.m$$

Joint	b	
member	ba	bc
D.f.	0.27	0.73
F.E.M.	24.07	35.00
Bal.M.	−15.95	−43.12
M_f	8.12	−8.12

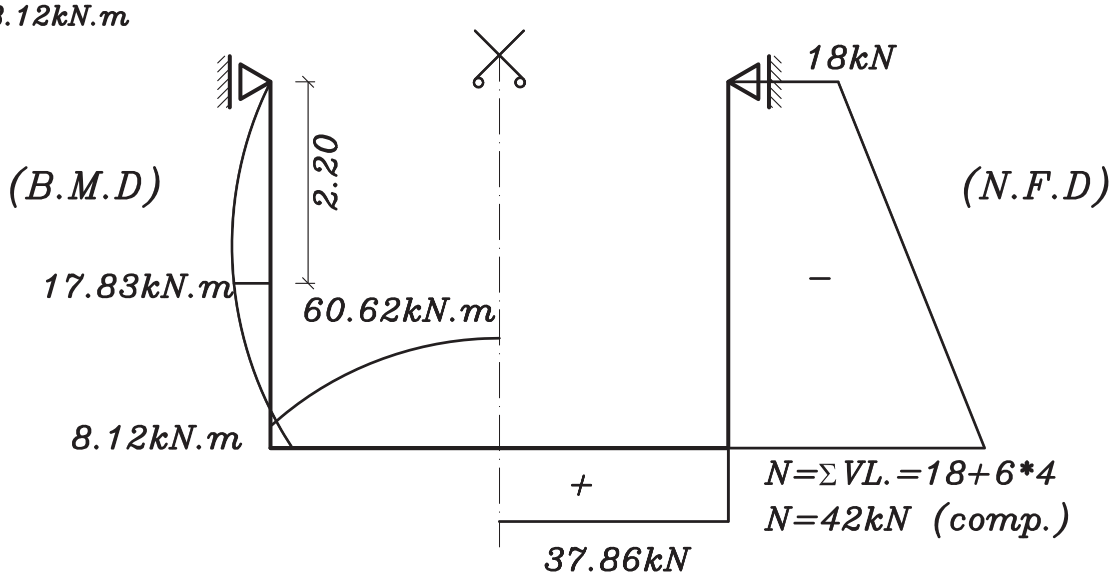


Point of zero shear

$$[\frac{x}{4}*20]*\frac{x}{2}=12.14 \Longrightarrow x=2.20m$$

$$\Longrightarrow M_{+ve}=12.14x-(\frac{x}{4}*20)*(\frac{x^2}{6})$$

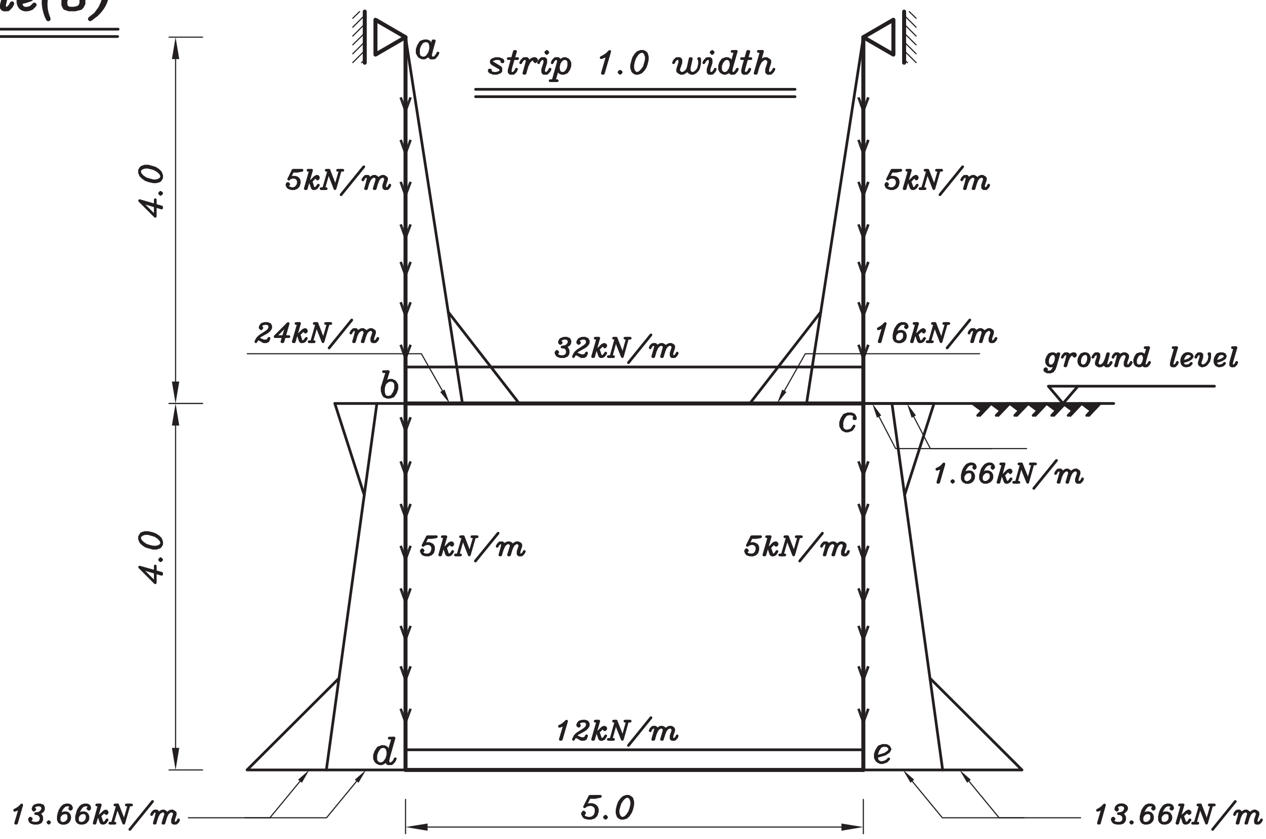
$$M_{+ve}=17.83\ kN.m$$



ـ ملحوظة هامة جدا

لاحظ اننا لم نعكس رد فعل الارضية على حائط الخزان و انما جمعنا جميع الاوزان الراسية على الحائط من اعلى لاسفل و ذلك لانه فى حالة (rested & underground tanks) تكون ارضية الخزان مرتكزة على (soil) و ليست على حائط الخزان .

Example(8)



- هذا الخزان يمثل خزان جزء منه فوق سطح الارض و الجزء الاخر تحت سطح الارض و بالتالى فان هذا الخزان ككل يكون مرتكز على الارضية (de)

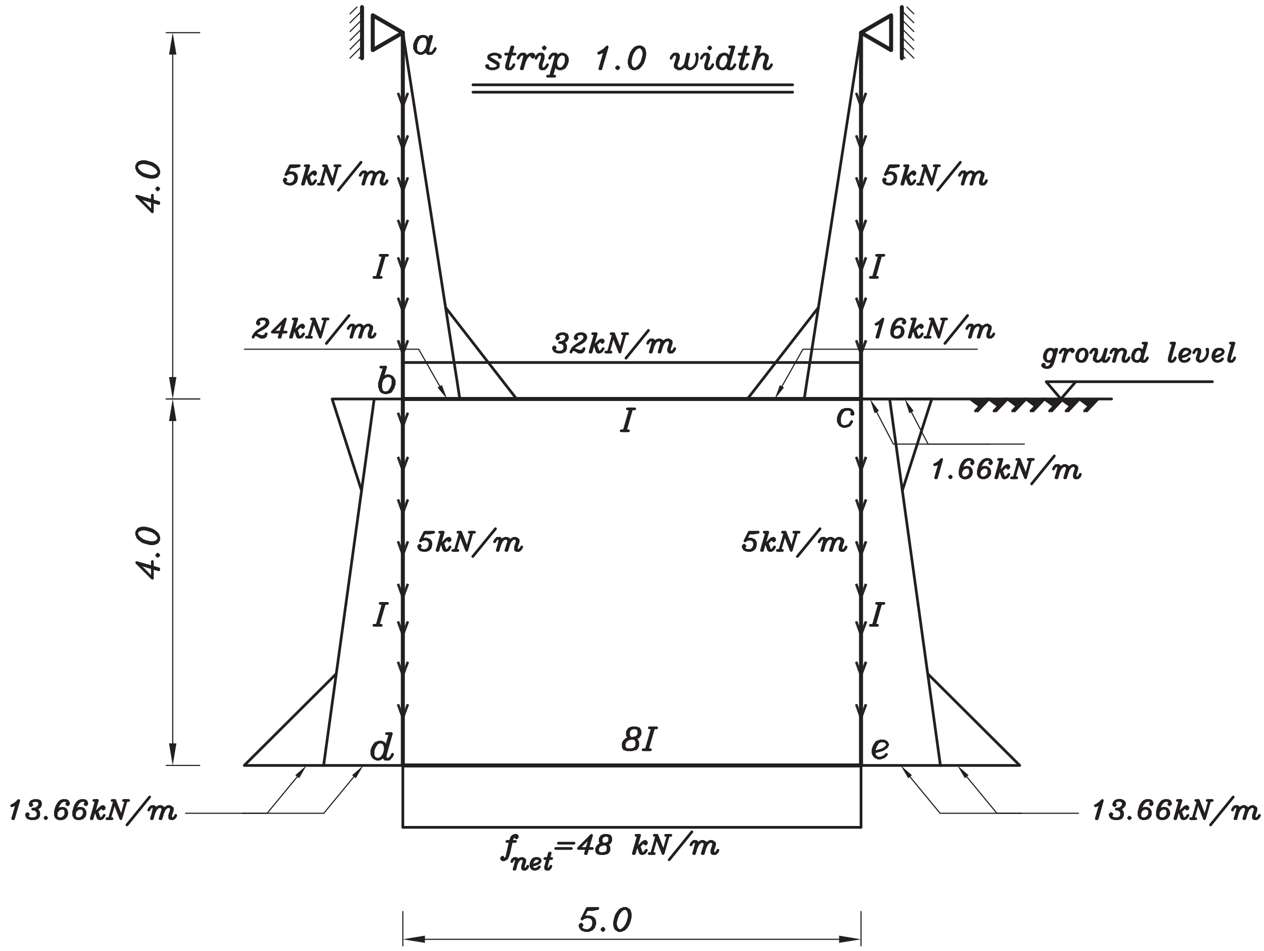
لذلك سوف نفرض ان $[I_{base\,(de)}=8I_{wall}=8I_{(bc)}]$

- الضغط الخارجى الواقع على حوائط الخزان يمثل ضغط التربة الجانبى (earth pressure)

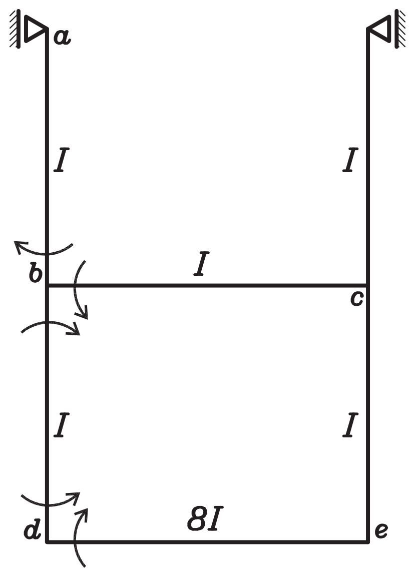
$$W_{total}=\underbrace{5*4*4}_{\text{o.w. of wall}}+\underbrace{32*5}_{\text{upper floor}}+\underbrace{12*5}_{\text{lower floor}}=300\ kN$$

$$f_{gross}=\frac{W_{total}}{A}=\frac{300}{5*1}=60\ kN/m$$

$$f_{net}=60\uparrow-12\downarrow=48\ kN/m\uparrow$$



For Joint b



$$D.f_{ba}=\frac{0.75(I/4.0)}{0.75(I/4.0)+0.5(I/5)+(I/4.0)}=0.35$$

$$D.f_{bc}=\frac{0.5(I/5.0)}{0.75(I/4.0)+0.5(I/5)+(I/4.0)}=0.19$$

$$D.f_{bd}=\frac{(I/4.0)}{0.75(I/4.0)+0.5(I/5)+(I/4.0)}=0.46$$

For Joint d

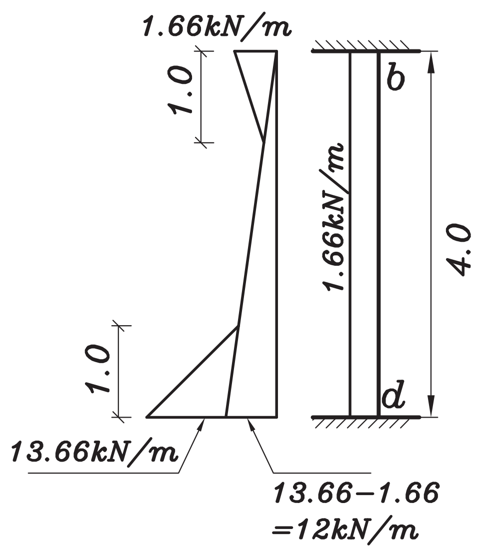
$$D.f_{db}=\frac{(I/4.0)}{(I/4.0)+0.5(8I/5)}=0.24$$

$$D.f_{de}=\frac{0.5(8I/5)}{(I/4.0)+0.5(8I/5)}=0.76$$

$$F.E.M._{ba}=\frac{24.0*(4)^2}{15}+\frac{16.0*(4)^2}{117}=27.79kN.m$$

$$F.E.M._{bc}=\frac{-32*(5.0)^2}{12}=-66.67kN.m$$

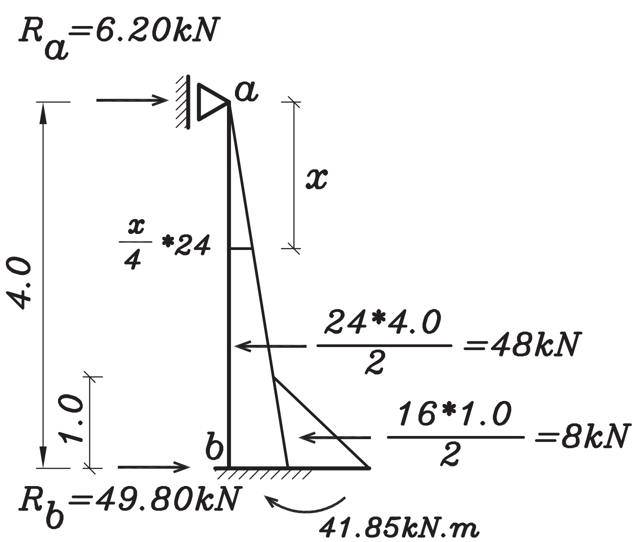
$$F.E.M._{bd} = \frac{1.66*(4)^2}{12} + \frac{12*(4)^2}{30} + \frac{1.66*(4)^2}{124} + \frac{13.66*(4)^2}{904} = 9.07 kN.m$$



$$F.E.M._{db} = \frac{-1.66*(4)^2}{12} - \frac{12*(4)^2}{20} - \frac{1.66*(4)^2}{904} - \frac{13.66*(4)^2}{124} = -13.61 kN.m$$

$$F.E.M._{de} = \frac{48*(5.0)^2}{12} = 100.0 kN.m$$

Joint	b			d	
member	ba	bc	bd	db	de
D.f.	0.35	0.19	0.46	0.24	0.76
F.E.M.	27.79	−66.67	9.07	−13.61	100
Bal.M.	10.43	5.66	13.71	−20.73	−65.66
C.O.M.	0	0	−10.37	6.86	0
Bal.M.	3.63	1.97	4.77	−1.65	−5.21
M_f	41.85	−59.04	17.18	−29.13	29.13

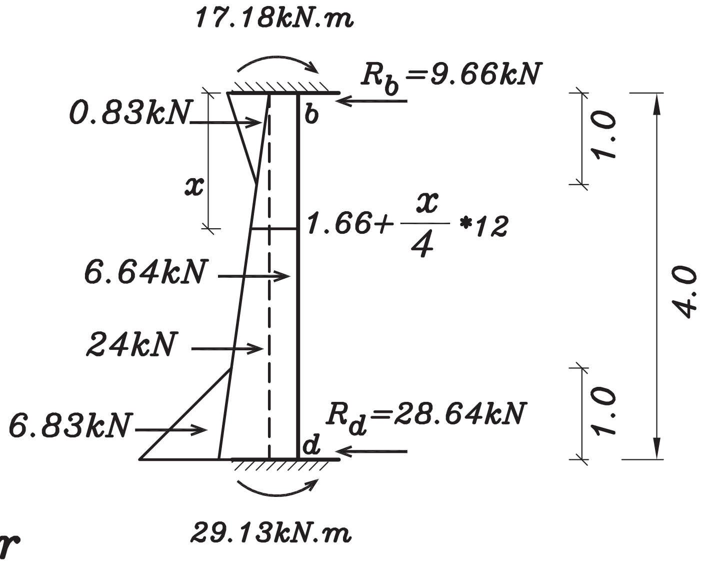


Point of zero shear

$$\left[\frac{x}{4}*24\right]*\frac{x}{2} = 6.20 \Longrightarrow x = 1.44 m$$

$$\Longrightarrow M_{+ve} = 6.20x - \left(\frac{x}{4}*24\right)*\left(\frac{x^2}{6}\right)$$

$$M_{+ve} = 5.94 \ kN.m$$

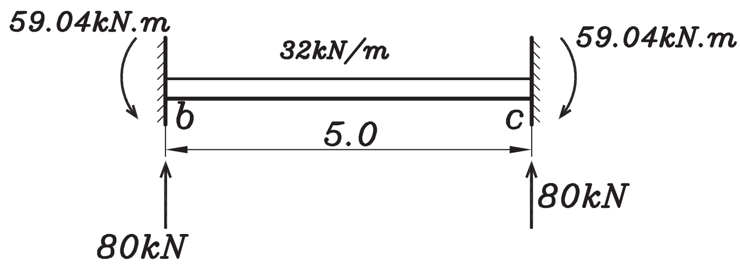


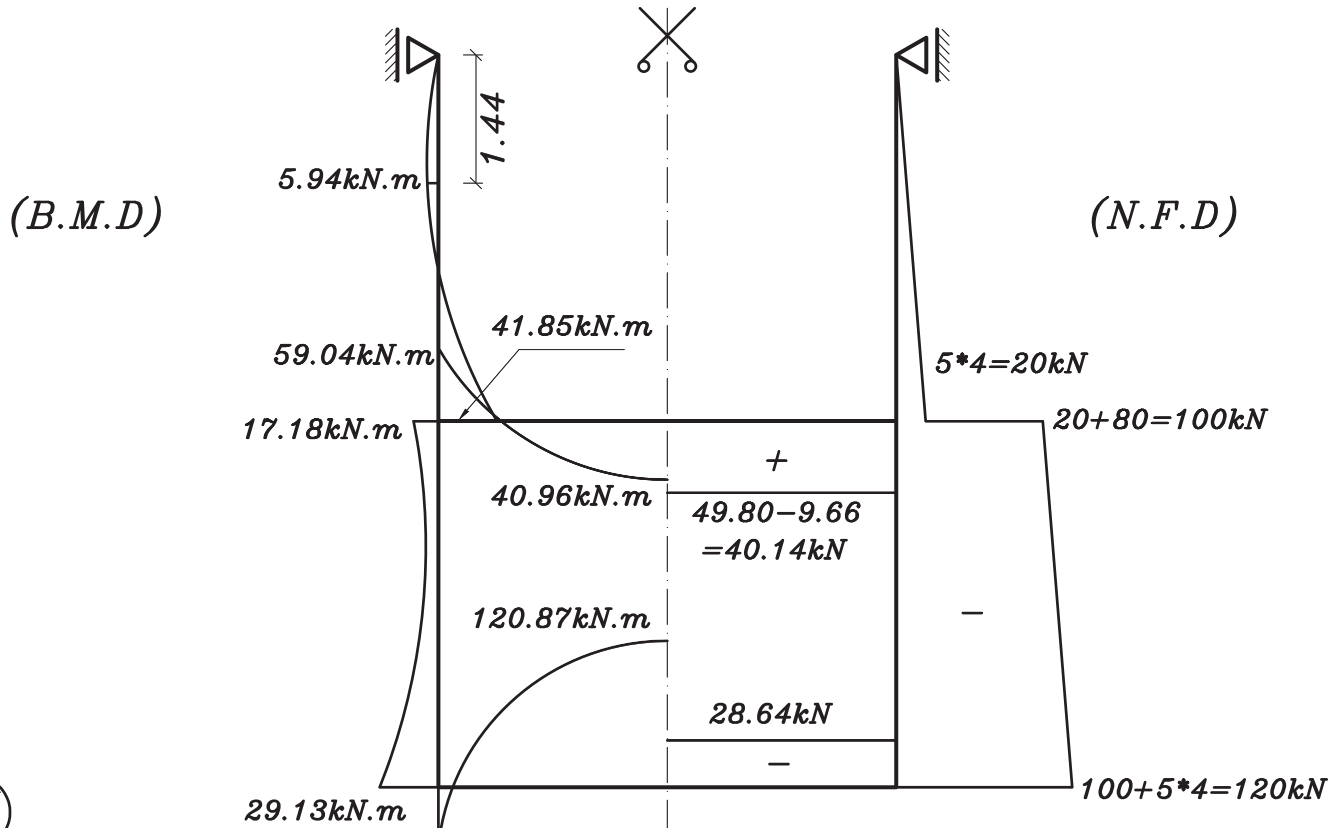
Point of zero shear

$$1.66x+[\frac{x}{4}*12]*\frac{x}{2}+0.83=9.66 \Longrightarrow x=1.94m$$

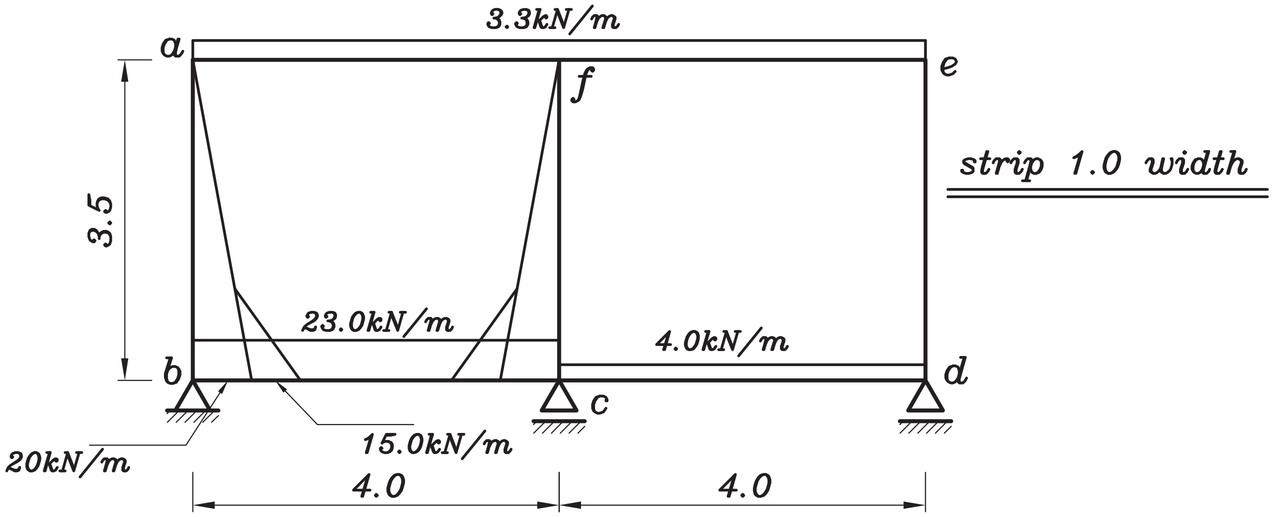
$$\Longrightarrow M_{+ve}=9.66x-1.66x\frac{x}{2}-(\frac{x}{4}*12)*(\frac{x^2}{6})$$

$$-0.83*(x-\frac{1}{3})-17.18=-6.55 \ \ kN.m$$





Example(9)



For Joint b, d, a, e

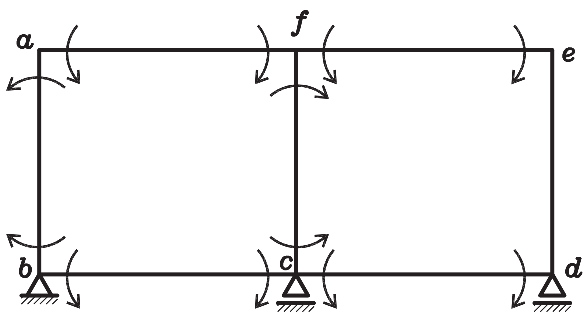
$$D.f_{ba}=D.f_{de}=D.f_{ab}=D.f_{ed}=\frac{(I/3.5)}{(I/3.5)+(I/4.0)}=0.53$$

$$D.f_{bc}=D.f_{dc}=D.f_{af}=D.f_{ef}=\frac{(I/4.0)}{(I/3.5)+(I/4.0)}=0.47$$

For Joint c, f

$$D.f_{cb}=D.f_{cd}=D.f_{fa}=D.f_{fe}=\frac{(I/4.0)}{2(I/4.0)+(I/3.5)}=0.32$$

$$D.f_{cf}=D.f_{fc}=\frac{(I/3.5)}{2(I/4.0)+(I/3.5)}=0.36$$



$$F.E.M._{ba}=\frac{20.0*(3.5)^2}{20}+\frac{15.0*(3.5)^2}{124}=13.73kN.m$$

$$F.E.M._{cf}=-13.73kN.m$$

$$F.E.M._{ab}=\frac{-20.0*(3.5)^2}{30}-\frac{15.0*(3.5)^2}{904}=-8.37kN.m$$

$$F.E.M._{fc}=8.37kN.m$$

$$F.E.M._{bc}=\frac{-23*(4.0)^2}{12}=-30.67kN.m$$

$$F.E.M._{cb}=30.67kN.m$$

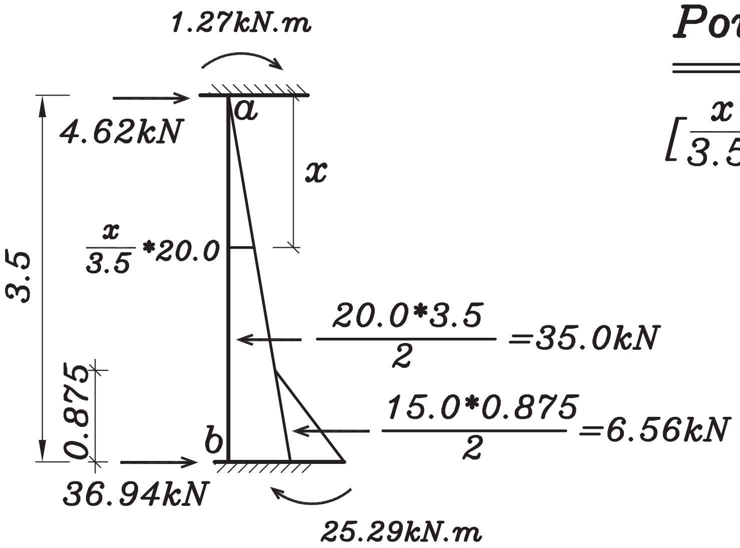
$$F.E.M._{cd}=\frac{-4*(4.0)^2}{12}=-5.33kN.m$$

$$F.E.M._{dc}=5.33kN.m$$

$$F.E.M._{\substack{af\\fe}}=\frac{-3.3*(4.0)^2}{12}=-4.4kN.m$$

$$F.E.M._{\substack{fa\\ef}}=4.4kN.m$$

Joint	a		b		c			d		e		f		
member	af	ab	ba	bc	cb	cf	cd	dc	de	ed	ef	fe	fc	fa
D.f.	0.47	0.53	0.53	0.47	0.32	0.36	0.32	0.47	0.53	0.53	0.47	0.32	0.36	0.32
F.E.M.	−4.40	−8.37	13.73	−30.67	30.67	−13.73	−5.33	5.33	0	0	4.40	−4.40	8.37	4.40
Bal.M.	6.00	6.77	8.98	7.96	−3.72	−4.18	−3.72	−2.51	−2.82	−2.33	−2.07	−2.68	−3.01	−2.68
C.O.M.	−1.43	4.49	3.39	−1.86	3.98	−1.51	−1.26	−1.86	−1.17	−1.41	−1.34	−1.04	−2.09	3.00
Bal.M.	−1.44	−1.62	−0.81	−0.72	−0.39	−0.44	−0.39	1.42	1.61	1.46	1.29	0.04	0.05	0.04
M_f	−1.27	1.27	25.29	−25.29	30.54	−19.86	−10.70	2.38	−2.38	−2.28	2.28	−8.08	3.32	4.76



Point of zero shear

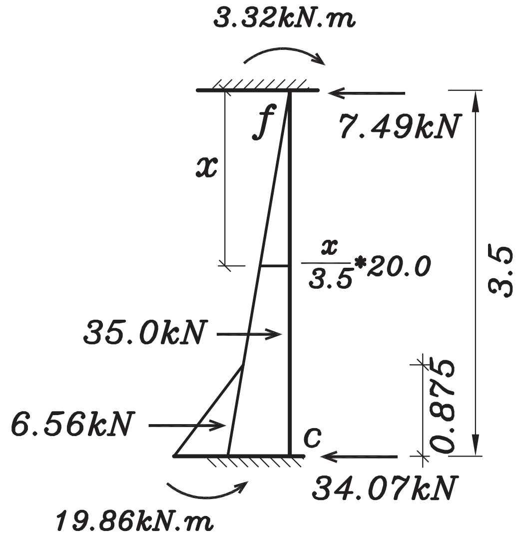
$$[\frac{x}{3.5}*20.0]*\frac{x}{2}=4.62 \Longrightarrow x=1.27m$$

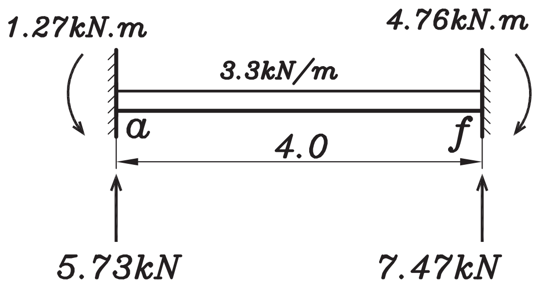
$$\Longrightarrow M_{+ve}=4.62x-(\frac{x}{3.5}*20.0)*(\frac{x^2}{6})+1.27=5.19 \;\; kN.m$$

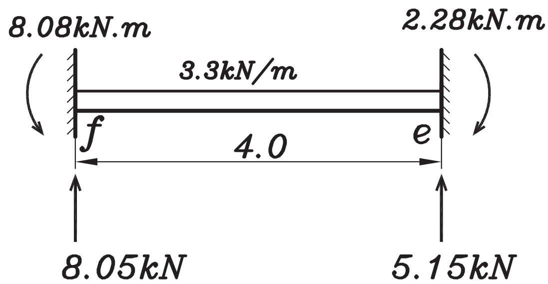
Point of zero shear

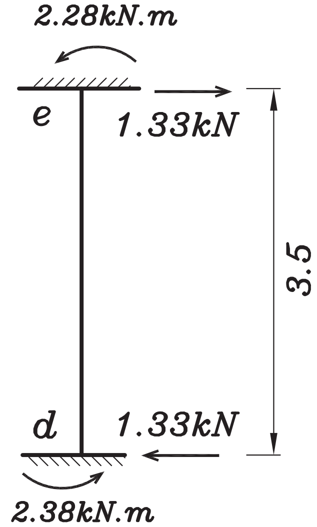
$$[\frac{x}{3.5}*20.0]*\frac{x}{2}=7.49 \Longrightarrow x=1.62m$$

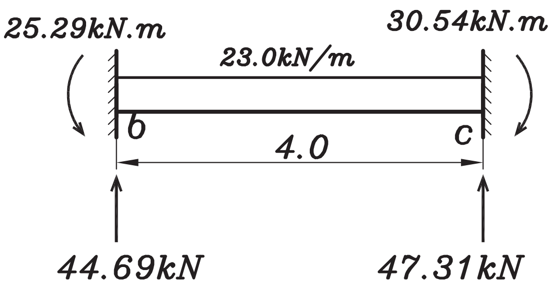
$$\Longrightarrow M_{+ve}=7.49x-(\frac{x}{3.5}*20.0)*(\frac{x^2}{6})-3.32=4.76 \;\; kN.m$$

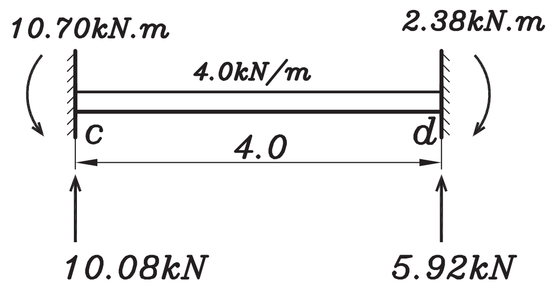


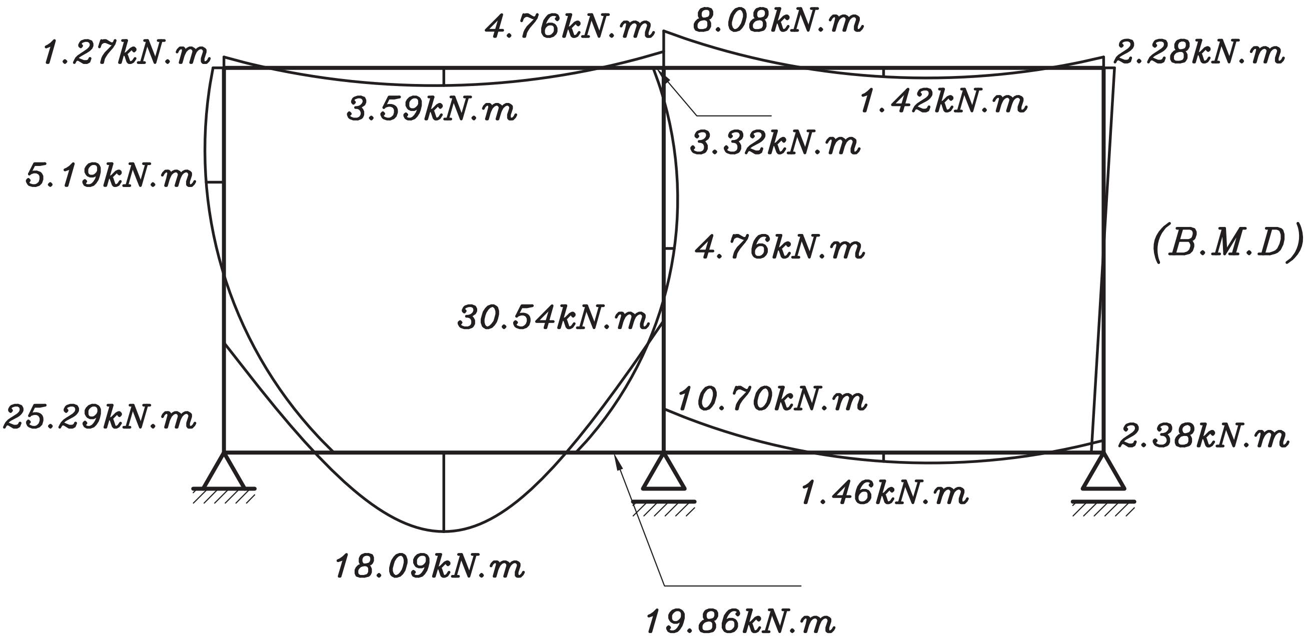


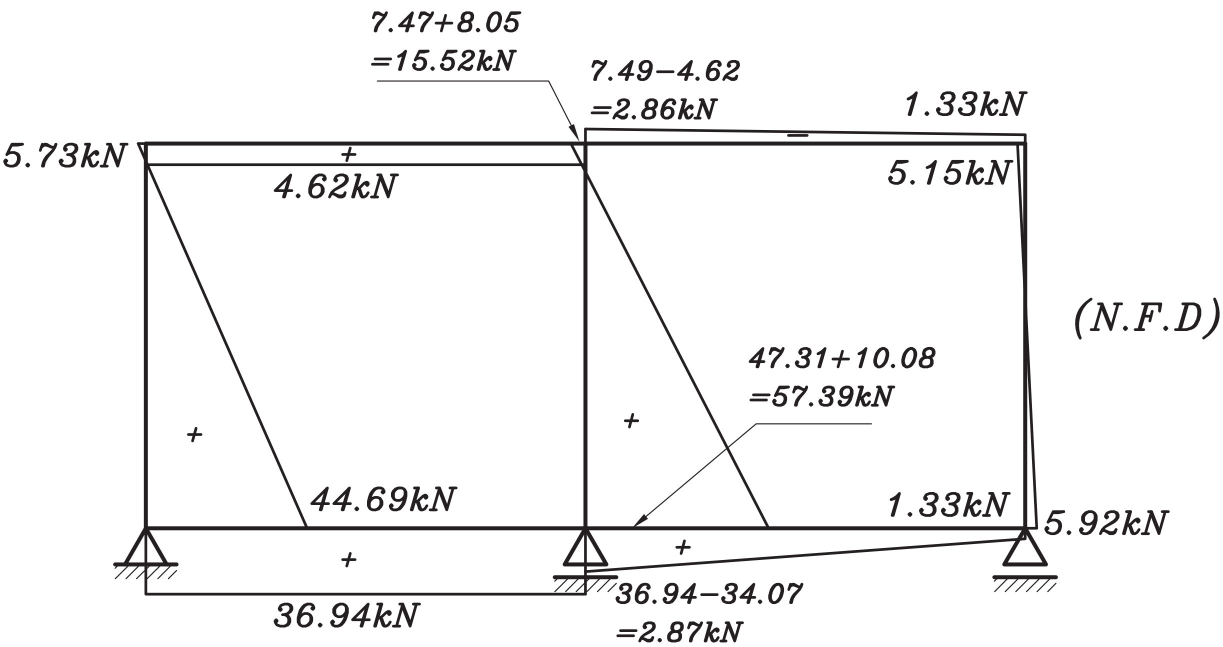






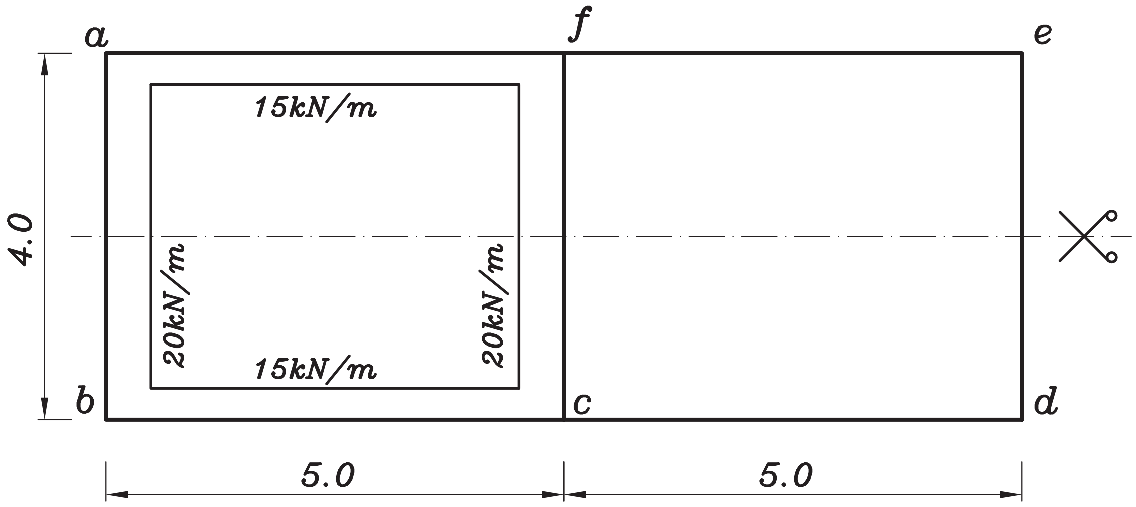



1.27kN.m
4.76kN.m
8.08kN.m
2.28kN.m
3.59kN.m
1.42kN.m
3.32kN.m
5.19kN.m
4.76kN.m
(B.M.D)
30.54kN.m
10.70kN.m
25.29kN.m
2.38kN.m
1.46kN.m
18.09kN.m
19.86kN.m


7.47+8.05
=15.52kN
7.49−4.62
=2.86kN
1.33kN
5.73kN
+
−
4.62kN
5.15kN
(N.F.D)
47.31+10.08
=57.39kN
+
+
44.69kN
1.33kN
5.92kN
+
+
36.94kN
36.94−34.07
=2.87kN

Example(10)

strip 1.0 width



For Joint b,d

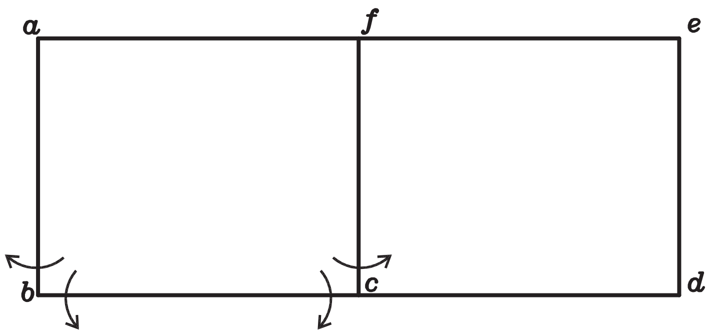
$$D.f_{ba}=D.f_{de}=\frac{0.5(I/4.0)}{0.5(I/4.0)+(I/5.0)}=0.38$$

$$D.f_{bc}=D.f_{dc}=\frac{(I/5.0)}{0.5(I/4.0)+(I/5.0)}=0.62$$

For Joint c

$$D.f_{cb}=D.f_{cd}=\frac{(I/5.0)}{2(I/5.0)+0.5(I/4.0)}=0.38$$

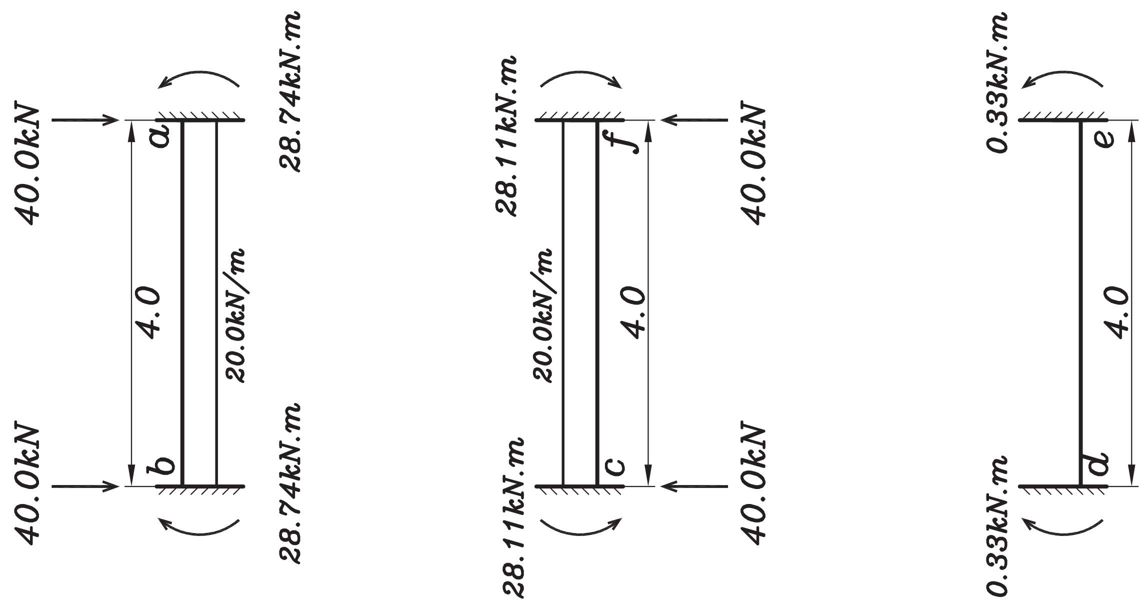
$$D.f_{cf}=\frac{0.5(I/4.0)}{2(I/5.0)+0.5(I/4.0)}=0.24$$

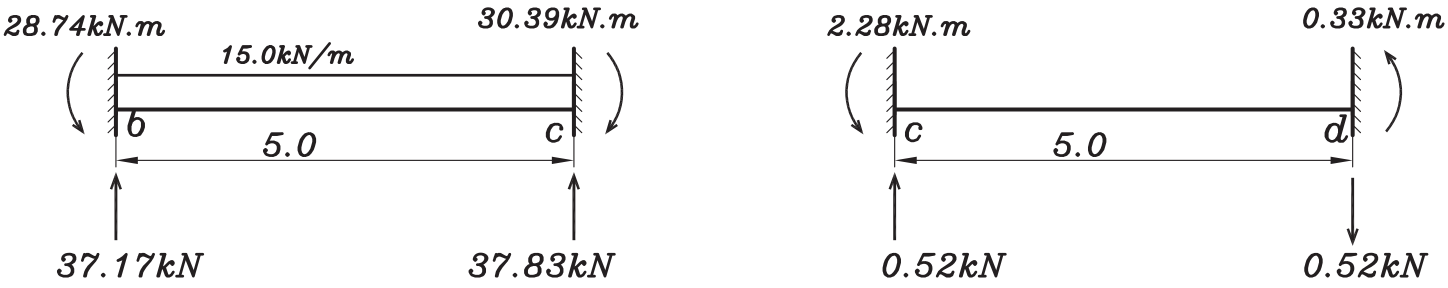


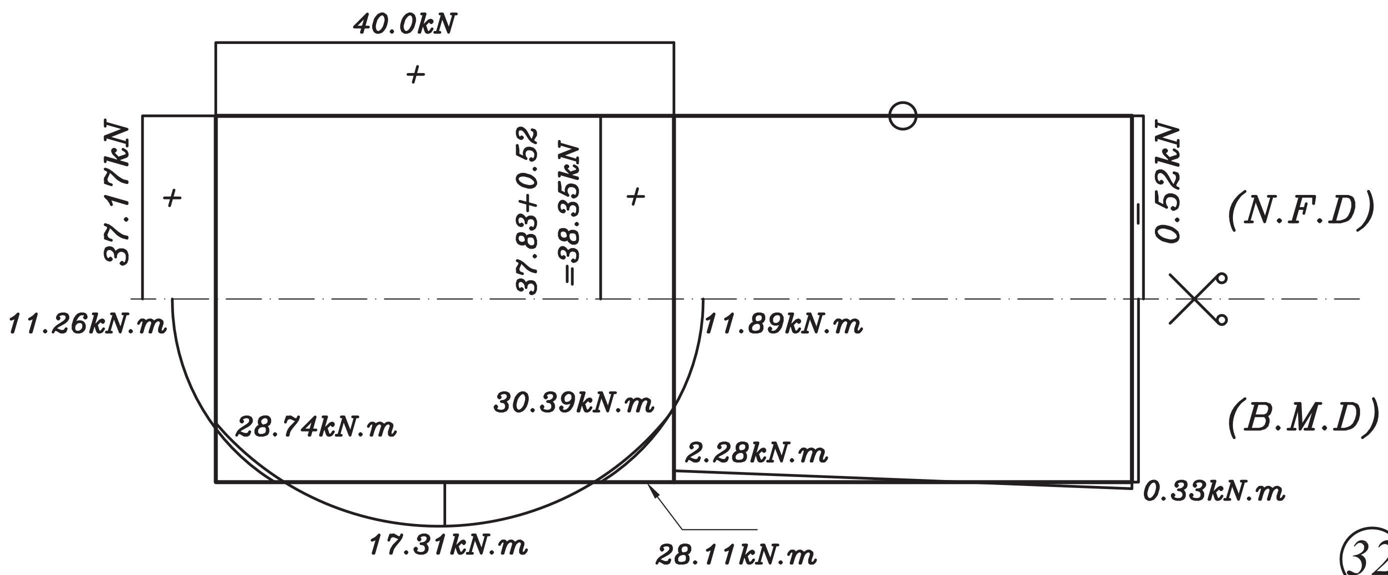
$$F.E.M._{bc}=\frac{-15.0*(5.0)^2}{12}=-31.25kN.m\ ,\ F.E.M._{cb}=31.25kN.m$$

$$F.E.M._{ba}=\frac{20.0*(4.0)^2}{12}=26.67kN.m\ ,\ F.E.M._{cf}=-26.67kN.m$$

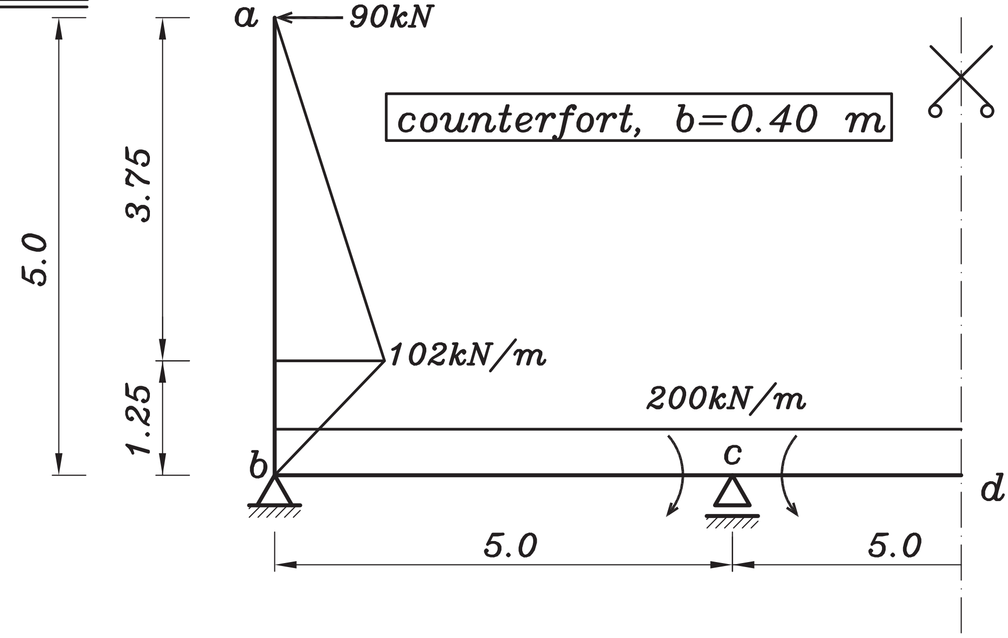
Joint	b		c			d	
member	ba	bc	cb	cf	cd	dc	de
D.f.	0.38	0.62	0.38	0.24	0.38	0.62	0.38
F.E.M.	26.67	−31.25	31.25	−26.67	0	0	0
Bal.M.	1.74	2.84	−1.74	−1.10	−1.74	0	0
C.O.M.	0	−0.87	1.42	0	0	−0.87	0
Bal.M.	0.33	0.54	−0.54	−0.34	−0.54	0.54	0.33
M_f	28.74	−28.74	30.39	−28.11	−2.28	−0.33	0.33







Example(11)



الشكل الوضح يمثل(counterfort)وهو عبارة عن(frame)يحمل داخله

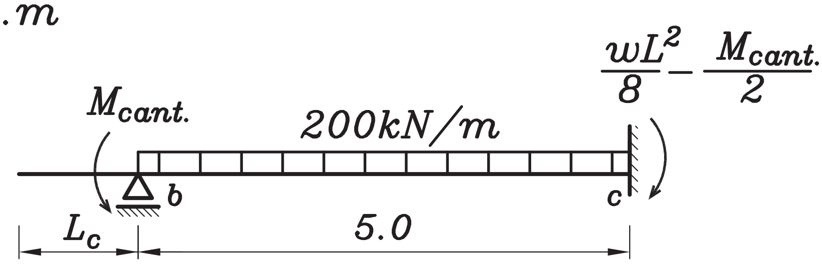
و(tank) و الركائز الموجودة عند (joints b,c) تمثل الاعمدة .

For Joint c

$$D.f_{cb} = \frac{0.75(I/5.0)}{0.75(I/5.0)+0.5(I/5)} = 0.60 \quad (fixed-hinged)$$

$$D.f_{cd} = \frac{0.5(I/5)}{0.75(I/5.0)+0.5(I/5)} = 0.40 \quad (symmetrical\ fixed-fixed)$$

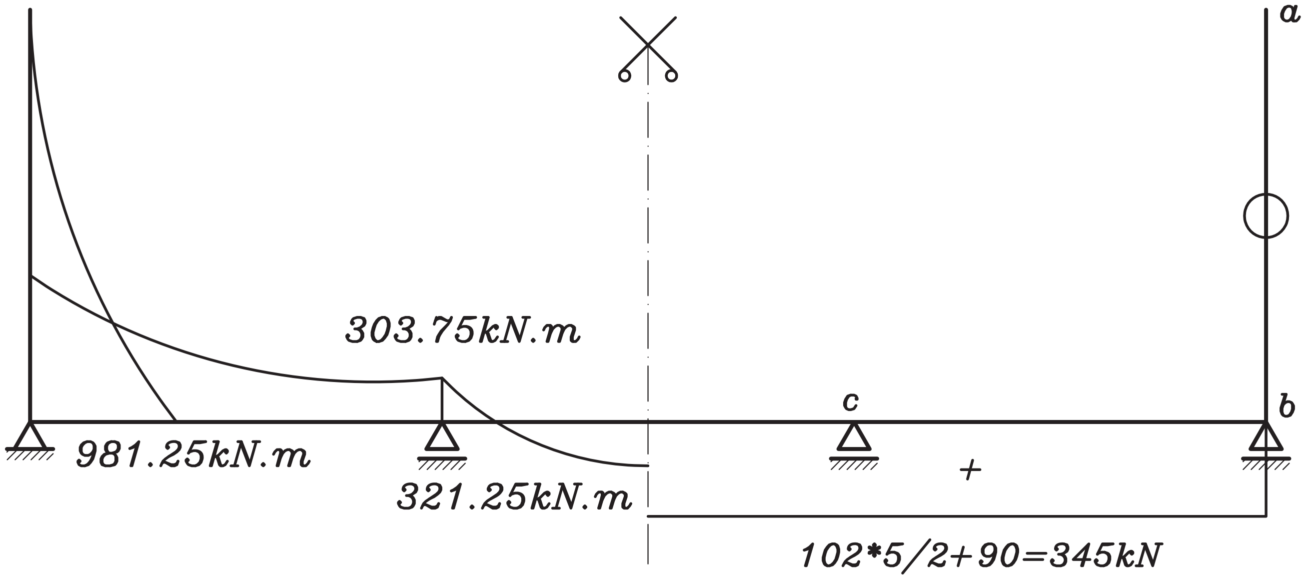
$$M_{ba} = M_{cant.} = \frac{102*3.75}{2}*(1.25+3.75/3)+\frac{102*1.25}{2}*(2/3*1.25)$$
$$+90*5 = 981.25kN.m$$



$$F.E.M._{cb} = \frac{200*(5)^2}{8} - \frac{981.25}{2} = 134.38kN.m$$

$$F.E.M._{cd} = \frac{-200*(5)^2}{12} = -416.67kN.m$$

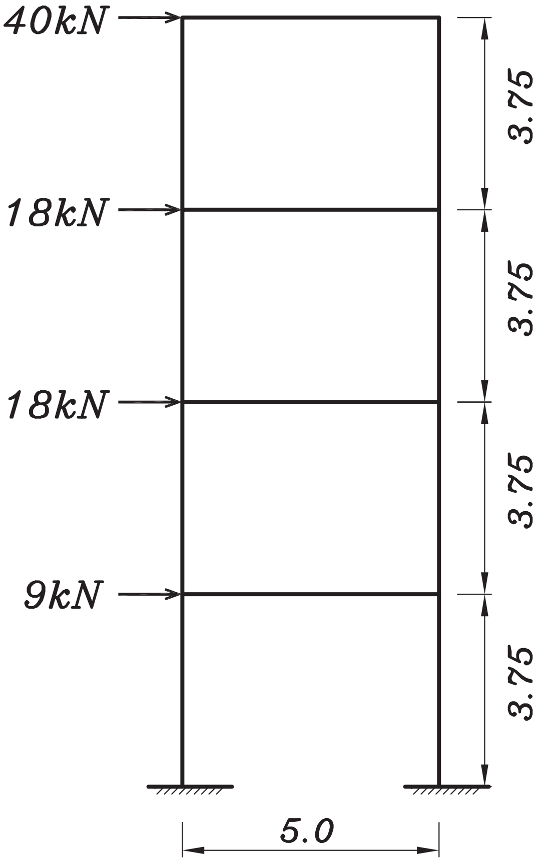
Joint	c	
member	cb	cd
D.f.	0.60	0.40
F.E.M.	134.38	−416.67
Bal.M.	169.37	112.92
M_f	303.75	−303.75



ـ ملحوظة هامة

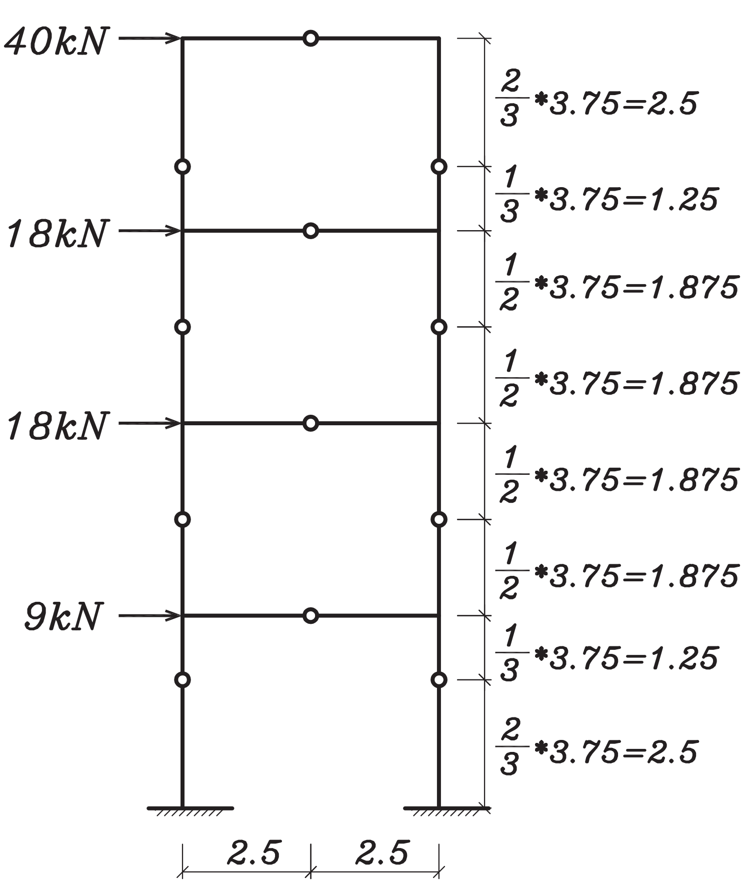
لاحظ ان $(normal\ \ force)$ على (ab) تساوى صفر لانه فى حالة $(counterfort)$ ينتقل رد فعل (bc) الى العمود مثل اى $(frame)$

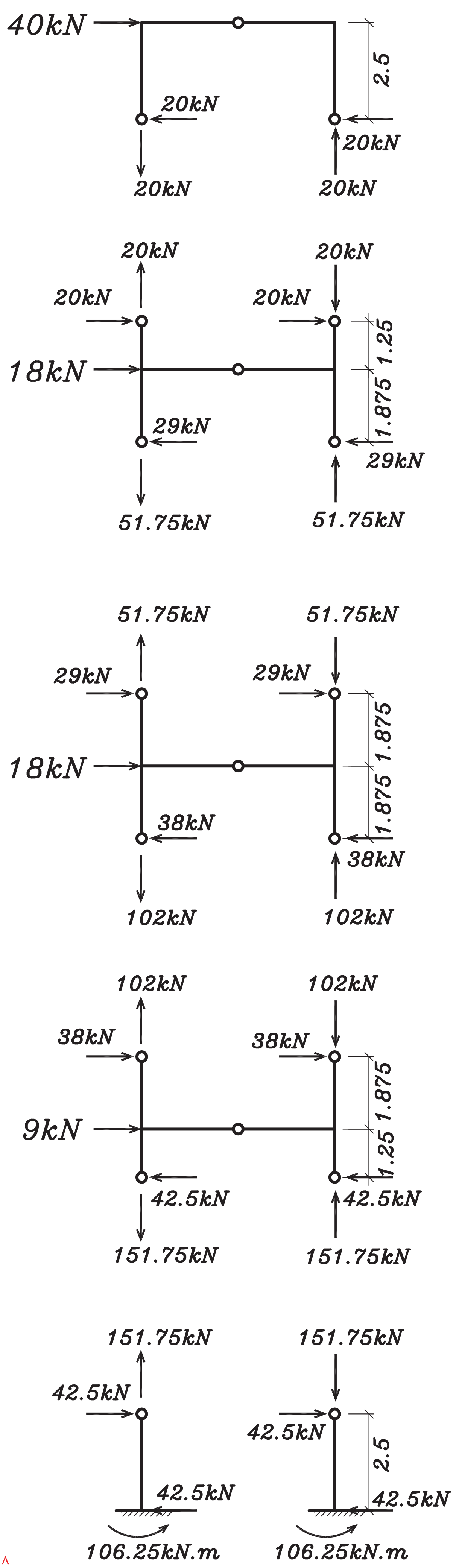
Example(12)

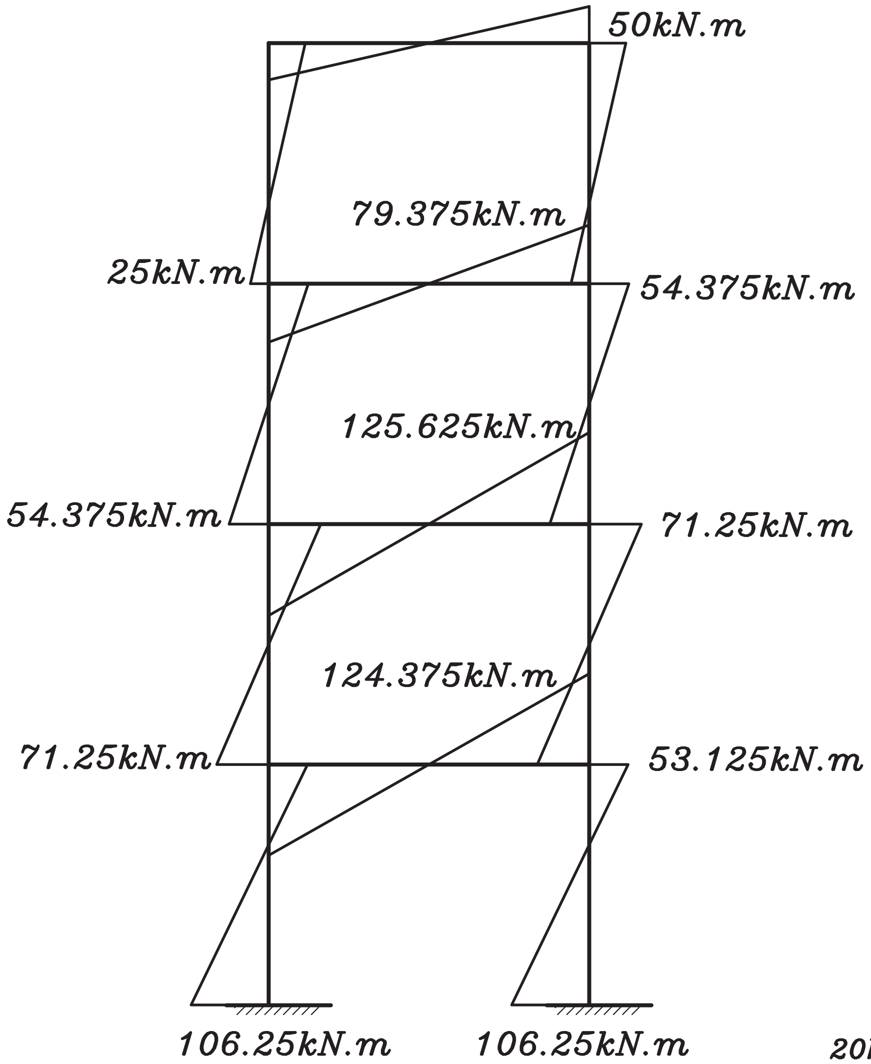


-هذا الـ $(frame)$ يمثل مجموعة الكمرات و الاعمدة التى تحمل $(tower\ \ tank)$ والقوى الافقية المؤثرة عليه تمثل احمال الرياح $(wind\ \ loads)$

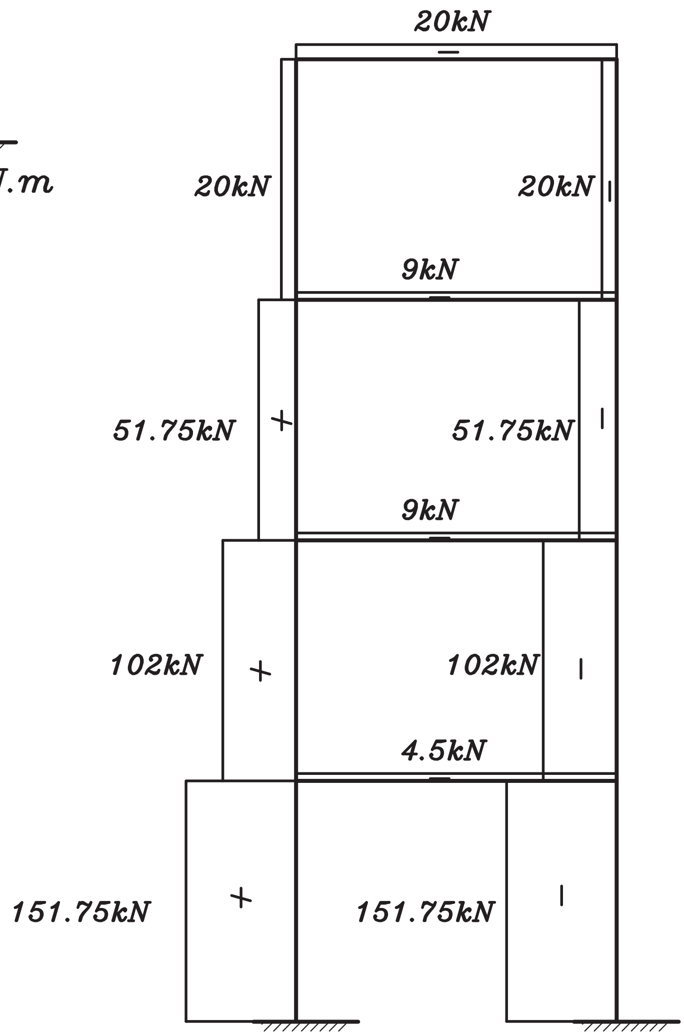
-نظرا لصعوبة حل هذا الـ $(frame)$ باستخدام $(moment\ \ distribution)$ فاننا نلجا لاستخدام طريقة تقريبية عن طريق فرض $(intermediate-hinges)$ عند الاماكن التى سيكون عندها $(B.M.=0)$


40kN
18kN
18kN
9kN
2/3 *3.75=2.5
1/3 *3.75=1.25
1/2 *3.75=1.875
1/2 *3.75=1.875
1/2 *3.75=1.875
1/2 *3.75=1.875
1/3 *3.75=1.25
2/3 *3.75=2.5
2.5
2.5


40kN
2.5
20kN
20kN
20kN
20kN
20kN
20kN
20kN
20kN
18kN
1.25
1.875
29kN
29kN
51.75kN
51.75kN
51.75kN
51.75kN
29kN
29kN
18kN
1.875
1.875
38kN
38kN
102kN
102kN
102kN
102kN
38kN
38kN
9kN
1.875
1.25
42.5kN
42.5kN
151.75kN
151.75kN
151.75kN
151.75kN
42.5kN
42.5kN
2.5
42.5kN
42.5kN
106.25kN.m
106.25kN.m


50kN.m
79.375kN.m
25kN.m
54.375kN.m
125.625kN.m
54.375kN.m
71.25kN.m
124.375kN.m
71.25kN.m
53.125kN.m
106.25kN.m
106.25kN.m

(B.M.D)


20kN
20kN
20kN
9kN
51.75kN
+
51.75kN
9kN
102kN
+
102kN
4.5kN
151.75kN
+
151.75kN

(N.F.D)

Design for crack control

-تتعرض المنشات الخرسانية الى مجموعة من العوامل البيئية الضارة مثل الرطوبة و الاملاح و هذه العوامل تؤثر بالضرر على حديد التسليح ٠
لذلك تم تقسيم المنشات الخرسانية المسلحة حسب تعرض اسطح الشد فيها لهذه العوامل الى

– Category one (protected tension side)

منشات ذات اسطح شد محمية

مثل جميع العناصر الداخلية من المنشات العادية و الاسقف النهائية المعزولة جيدا ٠

– Category two (unprotected tension side)

منشات ذات اسطح شد غير محمية

مثل الاسقف الداخلية و النهائية فى المناطق المجاورة للشواطئ و كذلك المنشات فى المناطق المفتوحة مثل الكبارى و الجراجات و الصالات المفتوحة و الاسقف النهائية الغير المعزولة جيدا ٠

– Category three (severly exposed tension side)

منشات ذات اسطح شد معرضة لعوامل ضارة

مثل خزانات المياه و الكبارى اعلى المجارى المائية (معرضة لرطوبة عالية) و المنشات المعرضة الى بخار الماء ٠

– Category four (very severly exposed tension side)

منشات ذات اسطح شد معرضة لعوامل تسبب صدا حديد التسليح

مثل المصانع الكيماوية و خزانات الصرف الصحى و المنشات المعرضة الى ماء البحر ٠

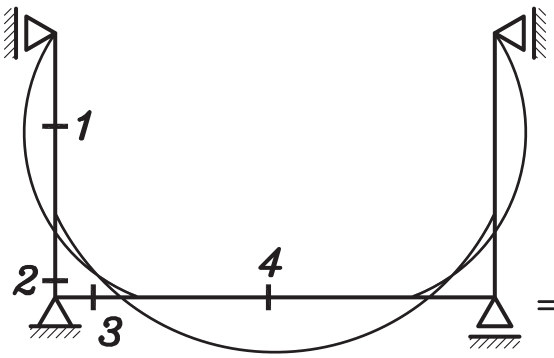
و عند تصميم المنشا الخرسانى يجب التاكد من ان عرض الشرخ المتولد لن يسمح بصدا حديد التسليح .

Crack width ≯

- 0.30 mm for category one
- 0.20 mm for category two
- 0.15 mm for category three
- 0.10 mm for category four

– Classification of R.C. sections accordindg to exposure of tension side to water:

انواع القطاعات الخرسانية طبقا لتعرض اسطح الشد بها للمياه



B.M.D. of strip in a water tank

Sec (1–1)

Tension side is out side the water ⟹ **Air section** ⟹ **Category two**

Sec (2–2) & (3–3)

Tension side is inside the water ⟹ **Water section** ⟹ **Category three**

Sec (4–4)

Tension side is out side the water ⟹ **Air section** ⟹ **Category two**

وبالتالى فان عند تصميم (sec 2,3) يجب التاكد من عدم تشرخ القطاع (uncracked section) حتى نضمن عدم وصول الماء الى حديد التسليح و بالتالى نضمن عدم صدا الحديد .

Factors affecting crack width

1– Concrete cover يقل عرض الشرخ كلما زاد سمك الغطاء الخرسانى

2– Types of steel used (steel 360/520 is better than steel 240/350)

حديد (٥٢/٣٦) يكون مشرشر بينما حديد (٣٥/٢٤) يكون املس و بالتالى تماسكه مع الخرسانة يكون اكبر فيقل عرض الشرخ .

3− Diameter of steel bar كلما زاد قطر السيخ يزداد عرض الشرخ

(if $A_s = 1100 mm^2 \Longrightarrow$ choose $A_s = 6\phi16$ is better than $4\phi19$)

و ذلك لان المساحة السطحية لـ $(6\phi16)$ اكبر من $(4\phi19)$

4− Characteristic strength of concrete (f_{cu})

كلما زادت (f_{cu}) يزداد تماسك الخرسانة مع حديد التسليح فيقل عرض الشرخ

5− Stress in steel RFT. (f_s)

كلما زاد (f_s) يزداد عرض الشرخ

⟸ مما سبق يتضح انه للتحكم فى عرض الشرخ يجب اخذ العوامل الاتية فى الاعتبار

1− Concrete cover

قيمة الغطاء الخرسانى تعتمد على (f_{cu}) و كذلك على نوع العنصر الانشائى (بلاطة او كمرة) .

Category	All elements except R.C. walls & solid slabs		R.C. walls & solid slabs	
	$f_{cu} \le 25$ Mpa	$f_{cu} > 25$ Mpa	$f_{cu} \le 25$ Mpa	$f_{cu} > 25$ Mpa
One	25 mm	20 mm	20 mm	20 mm
Two	30 mm	25 mm	25 mm	20 mm
Three	35 mm	30 mm	30 mm	25 mm
Four	45 mm	40 mm	40 mm	35 mm

ـ ملحوظة

يجب الا يقل الغطاء الخرسانى عن قطر اكبر سيخ مستخدم فى التسليح .

2- Allowable stress in steel

يتم تخفيض الاجهادات المسموحة فى الحديد طبقا لـ
(category, largest bar diameter & type of steel used)

- For deformed high strength bars

f_s (N/mm^2)	Reduction factor (β_{cr})		Category one	Category two	Category three, four
	st. 360/520	st. 400/600	Largest bar diameter (mm)		
220	1.00	0.92	18	12	8
200	0.93	0.83	22	16	10
180	0.85	0.75	25	20	12
160	0.75	0.67	32	22	18
140	0.65	0.58	——	25	22
120	0.56	0.50	——	——	28

حيث (f_s) هو الاجهاد المسموح به فى حالة التصميم بطريقة
(working stress design method)

(β_{cr}) هو معامل التخفيض لـ (f_y) فى حالة التصميم بطريقة
(ultimate limit design method)

Example

$M_{u.l.}=200\ kN.m$, $b=300mm$, $d=700mm$, $f_{cu}=25\ N/mm^2$

st. 360/520 , category (2)

$$700=C_1\sqrt{\frac{200*10^6}{300*25}} \qquad C_1=4.29 \quad \& \quad J=0.81$$

$$A_s=\frac{1}{\beta_{cr}}*\frac{M_{u.l.}}{J.d.f_y}$$ assume $\phi 18$ used $\Longrightarrow$ $\beta_{cr}=0.85$

$$A_s=\frac{1}{0.85}*\frac{200*10^6}{0.81*700*360}=1149.00\ mm^2 \Longrightarrow 5\phi 18$$

3- Allowable tensile stress in concrete

فى حالة المنشآت (category 3,4) يجب التاكد من ان اجهادات الشد فى القطاع الخرسانى اقل من اجهاد الشد المسموح به فى الخرسانة وذلك حتى نمنع تشرخ القطاع و بالتالى تكون الخرسانة مانعة لنفاذية السوائل ووصولها لحديد التسليح .

The tensile strength of concrete $f_{ctr} = 0.6\sqrt{f_{cu}}$ N/mm^2

⟹ The allowable tensile stress in concrete $f_{ct} = \frac{f_{ctr}}{\eta}$

حيث (η) هو معامل تخفيض لاجهادات الشد يعتمد على تخانة القطاع الخرسانى (t_v)

Virtual thickness (t_v)	≤100mm	200mm	400mm	≥600mm
η	1.00	1.30	1.60	1.70

where $$t_v = t\left[1 \pm \frac{(f_{ct})_N}{(f_{ct})_M}\right]$$

$$(f_{ct})_N = \frac{N}{A} \qquad \& \qquad (f_{ct})_M = \frac{M.y}{I} = \frac{M}{Z}$$

+ve sign if (N) is tension & −ve sign if (N) is compression

ـ ملحوظة هامة

فى حالة القطاعات المعرضة الى (bending moment) فقط فان ($t_v = t$)

– Steps of design of uncracked section (water section)

يتم التصميم على مرحلتين

Stage (I)

get the concrete dimension ⟹ calculate $f_t = \pm \frac{N}{A} + \frac{M}{Z}$

⟹ check $f_t < f_{ct}$ (if $f_t > f_{ct}$ ⟹ increase dimension)

و هذه الخطوة لضمان ان القطاع غير مشرخ و بالتالى غير منفذ لماء .

Stage (II)

Calculate steel RFT. (by design equations)

سوف نقوم فى الحالات الاتية بتصميم القطاعات الخرسانية التى اسطح شدها معرضة للمياه (water sections) على مرحلتين (stage I,II) طبقا للحالات الاتية

1– Design of sections subjected to axial tension (T)

Stage (I)

$$b * t_{(mm)} = 1000 \ k \ T_{kN}$$

where

k : معامل يعتمد على (f_{cu}, f_s) ويؤخذ من الجدول الموضح

T = working axial force (kN)

و تقرب (t) لاقرب (50 mm) بالزيادة

specail case

For walls and slabs b=1000mm (شريحة عرضها ١ م)

$$\Longrightarrow \quad t_{(mm)} = k \ T_{kN}$$

f_{cu} (N/mm^2)	f_{cto} (N/mm^2)	f_s (N/mm^2)							$k_{av.}$
		∞	200	180	160	140	120	100	
17.5	0.84	1.19	1.14	1.13	1.13	1.12	1.11	1.09	1.13
20.0	0.90	1.11	1.06	1.06	1.05	1.04	1.03	1.01	1.05
22.5	0.96	1.04	0.99	0.99	0.98	0.97	0.96	0.94	0.98
25.0	1.02	0.98	0.93	0.92	0.92	0.91	0.90	0.88	0.92
27.5	1.08	0.93	0.88	0.87	0.86	0.85	0.84	0.83	0.87
30.0	1.14	0.88	0.83	0.82	0.81	0.81	0.79	0.78	0.82

ـ ملحوظة

للتسهيل ناخذ (k) المقابلة لـ $(f_s = \infty)$

Stage (II)

$$A_s = \frac{T_{u.l.}}{\beta_{cr}.(f_y/\gamma_s)}$$

Example (1)

$T_{working} = 150$ kN , $b = 1000mm$, $f_{cu} = 25$ N/mm^2

st. 360/520 , category (3)

Stage (I)

$t_{(mm)} = k \; T_{kN} = 0.98*150 = 147mm \Longrightarrow$ take $t = 150mm$

Stage (II)

$A_s = \frac{T_{u.l.}}{\beta_{cr}.(f_y/\gamma_s)}$ assume $\phi 12$ used $\Longrightarrow \beta_{cr} = 0.85$

$A_s = \frac{150*1.5*10^3}{0.85*360/1.15} = 845.59 \; mm^2/m \Longrightarrow$ use $8\phi 12/m$

Example (2)

$T_{working}=200\ kN$, $b=300mm$, $f_{cu}=25\ N/mm^2$

st. 360/520 , category (3)

Stage (I)

$b*t_{(mm)}=1000\ k\ T_{kN}$

$300\ t=1000*0.98*200 \Longrightarrow t=653.30\ mm$

$\Longrightarrow$ Take $t=700mm$

Stage (II)

$A_s=\dfrac{T_{u.l.}}{\beta_{cr}\cdot(f_y/\gamma_s)}$ assume $\phi16$ used $\Longrightarrow \beta_{cr}=0.75$

$A_s=\dfrac{200*1.5*10^3}{0.75*360/1.15}=1277.78\ mm^2 \Longrightarrow$ use $7\phi16$

2- Design of sections subjected to pure bending moment

Stage (I)

$$t_{(mm)}=\sqrt{\frac{M*10^6}{b*factor}}$$

where و تقرب (t) لاقرب $(50\ mm)$ بالزيادة

M = working bending moment (kN.m)

b = width of cross section (mm)

specail case

For walls and slabs b=1000mm (شريحة عرضها ١ م)

$$\Longrightarrow \boxed{t_{(mm)} = \sqrt{\frac{M*10^3}{factor}}}$$

f_{cu} (N/mm^2)	25			27.5			30		
t_{mm}	$f_{t_{all.}}$	$M_{des.}$	t_{mm}	$f_{t_{all.}}$	$M_{des.}$	t_{mm}	$f_{t_{all.}}$	$M_{des.}$	t_{mm}
200	2.3	15.3	$\sqrt{\frac{M*10^3}{0.38}}$	2.4	16.0	$\sqrt{\frac{M*10^3}{0.40}}$	2.5	16.6	$\sqrt{\frac{M*10^3}{0.41}}$
400	1.8	48.0	$\sqrt{\frac{M*10^3}{0.30}}$	1.9	50.6	$\sqrt{\frac{M*10^3}{0.31}}$	2.0	53.3	$\sqrt{\frac{M*10^3}{0.33}}$
600	1.7	102	$\sqrt{\frac{M*10^3}{0.28}}$	1.8	108	$\sqrt{\frac{M*10^3}{0.30}}$	1.9	114	$\sqrt{\frac{M*10^3}{0.31}}$

Stage (II)

$$d = C_1\sqrt{\frac{M_{u.l.}}{b*f_{cu}}} \quad \& \quad A_s = \frac{1}{\beta_{cr}} * \frac{M_{u.l.}}{J*d*f_y}$$

Example (3)

$M_{working} = 14$ kN.m , $b = 1000mm$, $f_{cu} = 25$ N/mm^2

st. 360/520 , category (3)

Stage (I)

$$t_{(mm)} = \sqrt{\frac{M*10^3}{factor}} = \sqrt{\frac{14*10^3}{0.38}} = 191.94mm$$

$\Longrightarrow$ Take $t = 200mm$

Stage (II)

$M_{u.l.}=1.5*14=21$ kN.m , assume cover=40mm

$$160=C_1\sqrt{\frac{21*10^6}{1000*25}} \qquad C_1=5.52 \quad \& \quad J=0.826$$

$$A_s=\frac{1}{\beta_{cr}}*\frac{M_{u.l.}}{J.d.f_y} \qquad \text{assume } \phi 12 \text{ used} \Longrightarrow \beta_{cr}=0.85$$

$$A_s=\frac{1}{0.85}*\frac{21*10^6}{0.826*160*360}=519.28\ mm^2/m' \Longrightarrow 5\phi 12/m'$$

Example (4)

$M_{working}=20$ kN.m , b=250mm , $f_{cu}=25\ N/mm^2$

st. 360/520 , category (3)

Stage (I)

$$t_{(mm)}=\sqrt{\frac{M*10^6}{b*factor}}=\sqrt{\frac{20*10^6}{250*0.28}}=534.52mm$$

$\Longrightarrow$ Take t =550mm

Stage (II)

$M_{u.l.}=1.5*20=30$ kN.m , assume cover=50mm

$$500=C_1\sqrt{\frac{30*10^6}{250*25}} \qquad C_1=7.2 \quad \& \quad J=0.826$$

$$A_s=\frac{1}{\beta_{cr}}*\frac{M_{u.l.}}{J.d.f_y} \qquad \text{assume } \phi 12 \text{ used} \Longrightarrow \beta_{cr}=0.85$$

$$A_s = \frac{1}{0.85} * \frac{30*10^6}{0.826*500*360} = 237.38 \ mm^2$$

$$A_{s_{min}} = \frac{1.1}{f_y} bd = \frac{1.1}{360} 250*500 = 381.94 mm^2 > A_s$$

$$A_{s_{min}} = \begin{cases} \frac{1.1}{f_y} bd = \frac{1.1}{360} 250*500 = 381.94 mm^2 \\ 1.3 A_{s_{req}} = 1.3*237.38 = 308.60 mm^2 \\ \frac{0.15}{100} bd = \frac{0.15}{100} 250*500 = 187.50 mm^2 \end{cases}$$

$$A_{s_{min}} = 308.60 mm^2 \implies 3\phi 12$$

3- Design of sections subjected to B.M. , Normal force

case (a) : Normal force is compression

Stage (I)

$$t_{(mm)} = \sqrt{\frac{M*10^6}{b*factor}} - 20mm$$

و تقرب (t) لاقرب $(50 \ mm)$ بالزيادة

Check stresses $\implies f_t = -\frac{N}{A} + \frac{M}{Z} \leq f_{ct}$

Stage (II)

$$e = \frac{M}{N} \implies \frac{e}{t} \begin{cases} \leq 0.5 \\ > 0.5 \end{cases}$$

$if \ \frac{e}{t} \leq 0.5 \implies$ use interaction diagram calculate $\frac{N_{u.l.}}{btf_{cu}}$ & $\frac{M_{u.l.}}{bt^2 f_{cu}}$ $\Longrightarrow$ get ρ

$$\Longrightarrow \boxed{A_s = A_{s'} = \rho * 10^{-4} * f_{cu} * b * t}$$

$$if\ \frac{e}{t} > 0.5 \Longrightarrow e_s = e + \frac{t}{2} - cover \Longrightarrow M_{us} = N_{u.l.} * e_s$$

$$d = C_1 \sqrt{\frac{M_{us}}{b * f_{cu}}} \quad \& \quad A_s = \frac{1}{\beta_{cr}} * [\frac{M_{us}}{J * d * f_y} - \frac{N_{u.l.}}{f_y / \gamma_s}]$$

case (b) : Normal force is tension

Stage (I)

$$t_{(mm)} = \sqrt{\frac{M * 10^6}{b * factor}} + 40mm$$

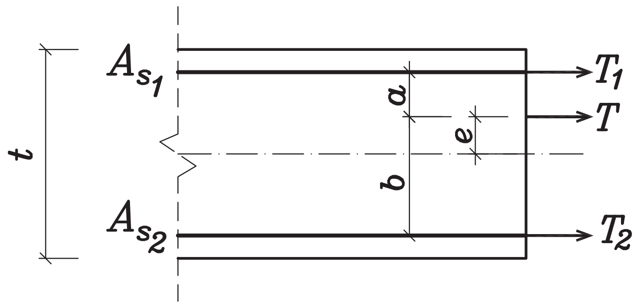
و تقرب (t) لاقرب $(50\ mm)$ بالزيادة

Check stresses

$$\Longrightarrow f_t = + \frac{T}{A} + \frac{M}{Z} \leq f_{ct}$$

Stage (II)

$$e = \frac{M}{T} \Longrightarrow e \begin{cases} \leq \frac{t}{2} - cover \\ > \frac{t}{2} - cover \end{cases}$$



$$if\ e \leq \frac{t}{2} - cover$$

$$\Longrightarrow a = \frac{t}{2} - e - cover \quad , \quad b = \frac{t}{2} + e - cover$$

$$\Longrightarrow T_1 = \frac{T.b}{a+b} \quad , \quad T_2 = \frac{T.a}{a+b}$$

$$\Longrightarrow A_{s_1} = \frac{T_{1\,u.l.}}{\beta_{cr} . (f_y / \gamma_s)} \quad , \quad A_{s_2} = \frac{T_{2\,u.l.}}{\beta_{cr} . (f_y / \gamma_s)}$$

$$if\ e > \frac{t}{2} - cover \Longrightarrow e_s = e - \frac{t}{2} + cover \Longrightarrow M_{us} = T_{u.l.} * e_s$$

$$d = C_1 \sqrt{\frac{M_{us}}{b * f_{cu}}} \quad \& \quad A_s = \frac{1}{\beta_{cr}} * [\frac{M_{us}}{J * d * f_y} + \frac{T_{u.l.}}{f_y / \gamma_s}]$$

Example (5)

$M_{working} = 30\ kN.m$, $N_{working} = 50\ kN\ (comp.)$, $b = 1000mm$

$f_{cu} = 25\ N/mm^2$, st. 360/520 , category (3)

Stage (I)

$$t_{(mm)} = \sqrt{\frac{M * 10^3}{factor}} - 20mm = \sqrt{\frac{30 * 10^3}{0.3}} - 20mm = 296.23mm$$

$\Longrightarrow$ Take $t = 300mm$

Check stresses

$$f_t = -\frac{N}{A} + \frac{M}{Z} = -\frac{50 * 10^3}{1000 * 300} + \frac{30 * 10^6}{1000 * (300)^2 / 6}$$

$$= -0.17 + 2 = 1.83\ N/mm^2$$

$$t_v = t[1 - \frac{(f_{ct})_N}{(f_{ct})_M}]$$

$$t_v = 300[1 - \frac{0.17}{2}] = 274.5mm \Longrightarrow f_{ct} = \frac{0.6\sqrt{25}}{1.6} = 1.88 N/mm^2$$

$\Longrightarrow f_t < f_{ct}$ (safe)

Stage (II)

$M_{u.l.} = 1.5 * 30 = 45\ kN.m$, $N_{u.l.} = 1.5 * 50 = 75\ kN$

$$e = \frac{M_{u.l.}}{N_{u.l.}} = \frac{45}{75} = 0.60m$$

$$\frac{e}{t} = \frac{0.60}{0.30} = 2.0 > 0.5\ (\text{ big eccentricity})$$

$$e_s = e + \frac{t}{2} - c = 0.60 + \frac{0.30}{2} - 0.04 = 0.71m$$

$M_{us}=75*0.71=53.25kN.m$

$d=300-40=260mm$

$$260=C_1\sqrt{\frac{53.25*10^6}{1000*25}} \qquad C_1=5.6 \quad \& \quad J=0.826$$

$$A_s=\frac{1}{\beta_{cr}}*[\frac{M_{us}}{J*d*f_y}-\frac{N_{u.l.}}{f_y/\gamma_s}]$$

assume $\phi 12$ used $\Longrightarrow$ $\beta_{cr}=0.85$

$$A_s=\frac{1}{0.85}[\frac{53.25*10^6}{0.826*260*360}-\frac{75*10^3}{360/1.15}]$$

$A_s=528.44mm^2/m'$ $\Longrightarrow$ $5\phi 12/m'$

Example (6)

$M_{working}=40$ kN.m , $T_{working}=40$ kN (ten.) , $b=1000mm$

$f_{cu}=22.5\ N/mm^2$, st. 360/520 , category (3)

Stage (I)

$$t_{(mm)}=\sqrt{\frac{M*10^3}{factor}}+40mm=\sqrt{\frac{40*10^3}{0.28}}+40mm=417.96mm$$

$\Longrightarrow$ Take $t=450mm$

Check stresses

$$f_t=+\frac{T}{A}+\frac{M}{Z}=+\frac{40*10^3}{1000*450}+\frac{40*10^6}{1000*(450)^2/6}$$

$$=0.09+1.19=1.28\ N/mm^2$$

$$t_v=t[1+\frac{(f_{ct})_N}{(f_{ct})_M}]$$

$$t_v=450[1+\frac{0.09}{1.19}]=484.03mm \Longrightarrow f_{ct}=\frac{0.6\sqrt{22.5}}{1.7}=1.67N/mm^2$$

$\Longrightarrow f_t < f_{ct}$ (safe)

Stage (II)

$$M_{u.l.}=1.5*40=60\ kN.m \quad , \quad T_{u.l.}=1.5*40=60\ kN$$

$$e=\frac{M_{u.l.}}{T_{u.l.}}=\frac{60}{60}=1.0m > \frac{t}{2}-cover \qquad (assuming\ cover=40mm)$$

$$e_s=e-\frac{t}{2}+c=1.00-\frac{0.45}{2}+0.04=0.82m$$

$$M_{us}=60*0.82=48.90kN.m$$

$$410=C_1\sqrt{\frac{48.90*10^6}{1000*22.5}} \qquad C_1=8.8 \quad \& \quad J=0.826$$

$$A_s=\frac{1}{\beta_{cr}}*[\frac{M_{us}}{J*d*f_y}+\frac{T_{u.l.}}{f_y/\gamma_s}]$$

assume $\phi 12$ used $\Longrightarrow$ $\beta_{cr}=0.85$

$$A_s=\frac{1}{0.85}[\frac{48.90*10^6}{0.826*410*360}+\frac{60*10^3}{360/1.15}]$$

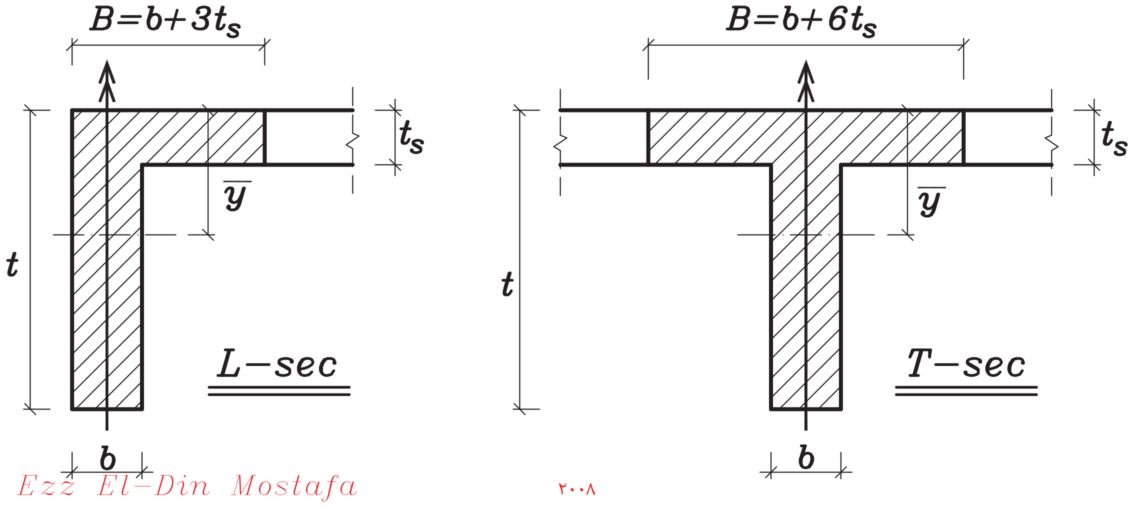
$$A_s=697.36mm^2/m' \Longrightarrow 7\phi 12/m'$$

ـ ملحوظة هامة

عند تصميم الكمرات (category 3,4) فانه يجب اتباع الاتى

case (a) : Flange is at compression side

Stage (I)



$$t_{(mm)} = \sqrt{\frac{M*10^6}{b*factor}} *(0.6-0.7)$$

و تقرب (t) لاقرب $(50\ mm)$

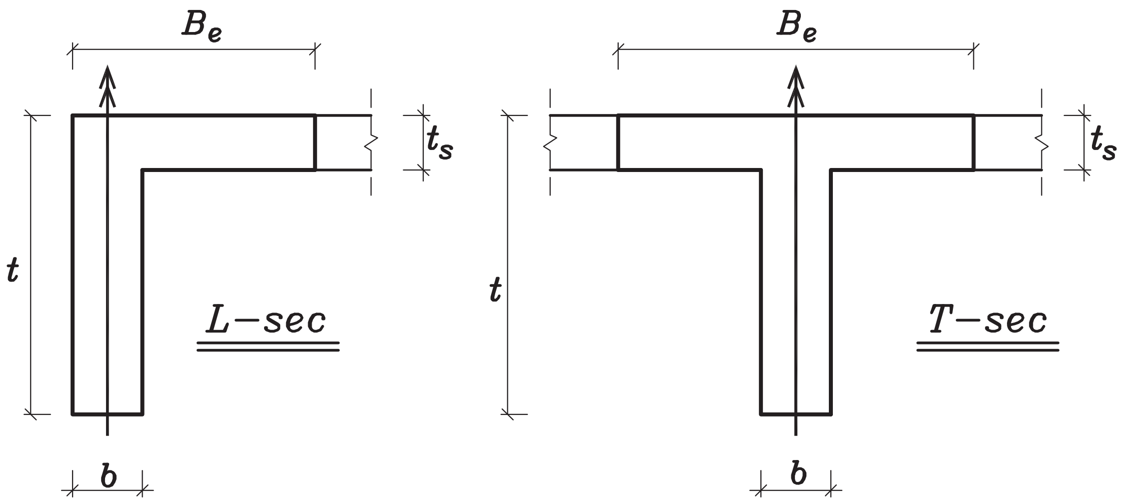
نقلل (t) لان $(inertia)$ للقطاعات $(T,L-sections)$ تكون اكبر من $(inertia)$ للقطاعات $(R-sections)$ التى لها نفس ال (b,t)

Calculate $\frac{t_s}{t}$ & $\frac{b}{B}$ $\Longrightarrow$ from tables get μ, η

$\Longrightarrow$ Get $\bar{y} = \eta t$, $I = (\mu * 10^{-4}) B t^3$, $A = B t_s + b(t - t_s)$

Check stresses $\Longrightarrow$ $f_t = \pm \frac{N}{A} + \frac{M(t-\bar{y})}{I} \leq f_{ct}$

Stage (II)

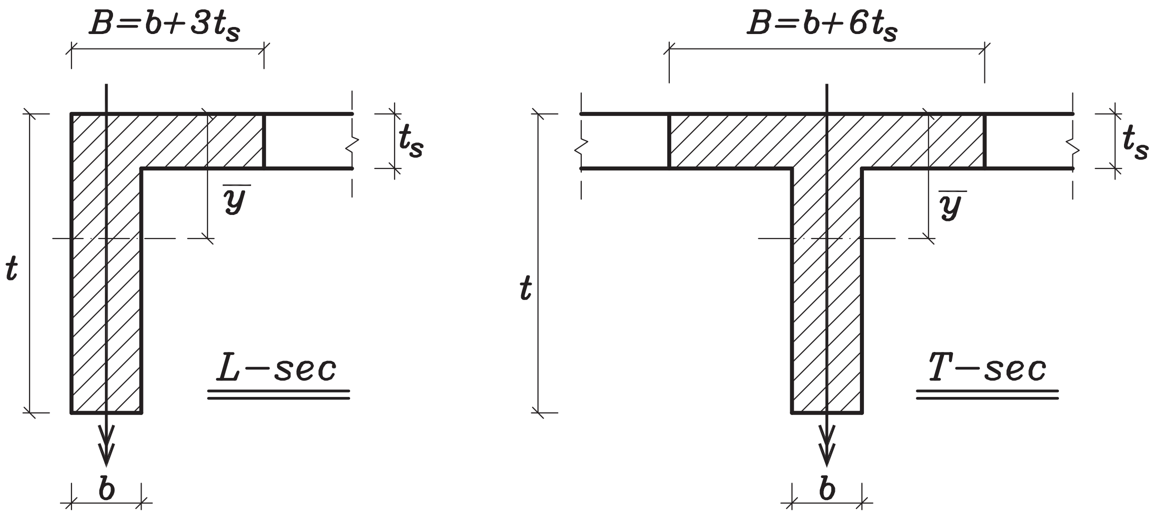


$$B_e = \begin{cases} 6t_s + b \\ C.L. \rightarrow C.L. \quad \text{الاقل} \\ \frac{KL}{10} + b \end{cases} \qquad B_e = \begin{cases} 16t_s + b \\ C.L. \rightarrow C.L. \quad \text{الاقل} \\ \frac{KL}{5} + b \end{cases}$$

$$d = C_1 \sqrt{\frac{M_{u.l.}}{B * f_{cu}}} \quad \& \quad A_s = \frac{1}{\beta_{cr}} * \frac{M_{u.l.}}{J * d * f_y}$$

case (b) : Flange is at tension side

Stage (I)



$$t_{(mm)} = \sqrt{\frac{M*10^6}{b*factor}} *(0.6-0.7)$$

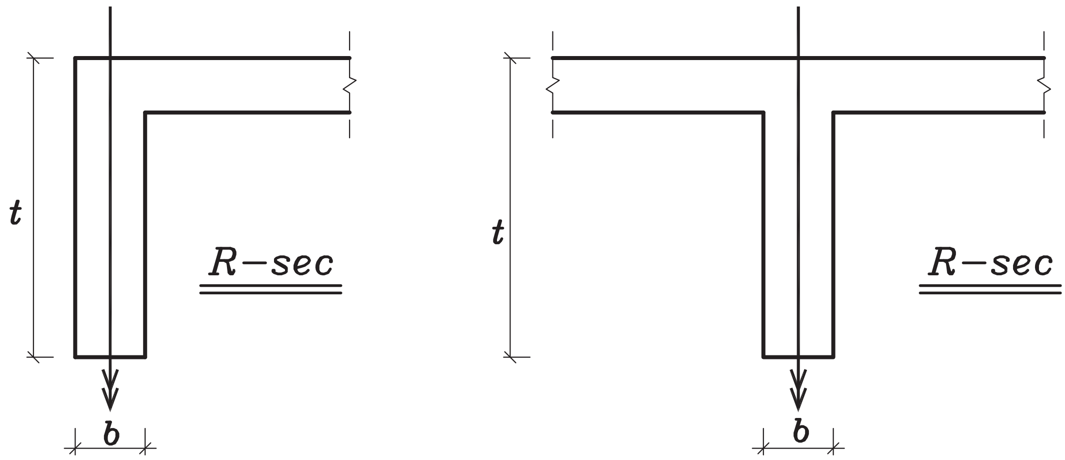
و تقرب (t) لاقرب $(50\ mm)$

Calculate $\frac{t_s}{t}$ & $\frac{b}{B}$ $\Longrightarrow$ from tables get μ, η

$\Longrightarrow$ Get $\bar{y} = \eta t$, $I = (\mu * 10^{-4}) B t^3$, $A = B t_s + b(t - t_s)$

Check stresses $\Longrightarrow$ $f_t = \pm \frac{N}{A} + \frac{My}{I} \leq f_{ct}$

Stage (II)



لاحظ ان ال$(flange)$ناحية الشد و بالتالى يكون القطاع $(R-sec)$

$$d = C_1 \sqrt{\frac{M_{u.l.}}{b*f_{cu}}} \quad \& \quad A_s = \frac{1}{\beta_{cr}} * \frac{M_{u.l.}}{J*d*f_y}$$

1- Design of sections subjected to axial tension (T)

Stage (I)

$$b*t_{(mm)}=1000\ k\ T_{kN}$$

specail case

For walls and slabs b=1000mm

$$t_{(mm)}=k\ T_{kN}$$

where

k : معامل يعتمد على (f_{cu},f_s) ويؤخذ من الجداول

T = working axial force (kN)

و تقرب (t) لاقرب (50 mm) بالزيادة

Stage (II)

$$A_s=\frac{T_{u.l.}}{\beta_{cr}.(f_y/\gamma_s)}$$

2- Design of sections subjected to pure bending moment

Stage (I)

$$t_{(mm)}=\sqrt{\frac{M*10^6}{b*factor}}$$

specail case

For walls and slabs b=1000mm

$$t_{(mm)}=\sqrt{\frac{M*10^3}{factor}}$$

where

M = working bending moment (kN.m)

Stage (II)

$$d=C_1\sqrt{\frac{M_{u.l.}}{b*f_{cu}}} \quad \& \quad A_s=\frac{1}{\beta_{cr}}*\frac{M_{u.l.}}{J*d*f_y}$$

3- Design of sections subjected to B.M. , Normal force

case (a) : Normal force is compression

Stage (I)

$$t_{(mm)} = \sqrt{\frac{M*10^6}{b*factor}} - 20mm$$

و تقرب (t) لاقرب $(50\ mm)$ بالزيادة

Check stresses $\Longrightarrow f_t = -\frac{N}{A} + \frac{M}{Z} \leq f_{ct}$

Stage (II)

$$e = \frac{M}{N} \Longrightarrow \frac{e}{t} \begin{cases} \leq 0.5 \\ > 0.5 \end{cases}$$

if $\frac{e}{t} \leq 0.5 \Longrightarrow$ use interaction diagram

calculate $\frac{N_{u.l.}}{btf_{cu}}$ & $\frac{M_{u.l.}}{bt^2 f_{cu}} \Longrightarrow$ get ρ

$$\Longrightarrow \boxed{A_s = A_{s'} = \rho * 10^{-4} * f_{cu} * b * t}$$

if $\frac{e}{t} > 0.5 \Longrightarrow e_s = e + \frac{t}{2} - cover \Longrightarrow M_{us} = N_{u.l.} * e_s$

$$d = C_1 \sqrt{\frac{M_{us}}{b*f_{cu}}} \quad \& \quad A_s = \frac{1}{\beta_{cr}} * \left[\frac{M_{us}}{J*d*f_y} - \frac{N_{u.l.}}{f_y / \gamma_s}\right]$$

case (b) : Normal force is tension

Stage (I)

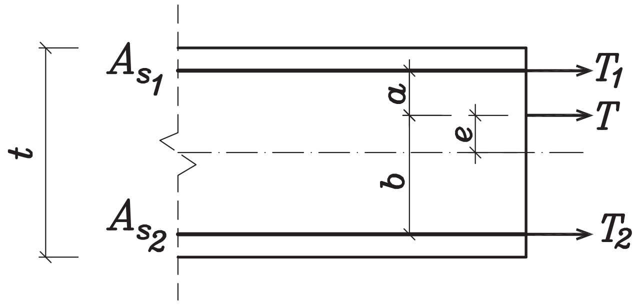
$$t_{(mm)} = \sqrt{\frac{M*10^6}{b*factor}} + 40mm$$

و تقرب (t) لاقرب $(50\ mm)$ بالزيادة

Check stresses $\Longrightarrow f_t = +\frac{T}{A} + \frac{M}{Z} \leq f_{ct}$

Stage (II)

$$e=\frac{M}{T} \Longrightarrow e \begin{cases} \le \frac{t}{2} - cover \\ > \frac{t}{2} - cover \end{cases}$$



if $e \le \frac{t}{2} - cover$

$$\Longrightarrow a=\frac{t}{2}-e-cover \quad , \quad b=\frac{t}{2}+e-cover$$

$$\Longrightarrow T_1=\frac{T.b}{a+b} \quad , \quad T_2=\frac{T.a}{a+b}$$

$$\Longrightarrow A_{s_1}=\frac{T_{1\,u.l.}}{\beta_{cr}.(f_y/\gamma_s)} \quad , \quad A_{s_2}=\frac{T_{2\,u.l.}}{\beta_{cr}.(f_y/\gamma_s)}$$

if $e > \frac{t}{2} - cover \Longrightarrow e_s = e - \frac{t}{2} + cover \Longrightarrow M_{us}=T_{u.l.}*e_s$

$$d=C_1\sqrt{\frac{M_{us}}{b*f_{cu}}} \quad \& \quad A_s=\frac{1}{\beta_{cr}}*[\frac{M_{us}}{J*d*f_y}+\frac{T_{u.l.}}{f_y/\gamma_s}]$$

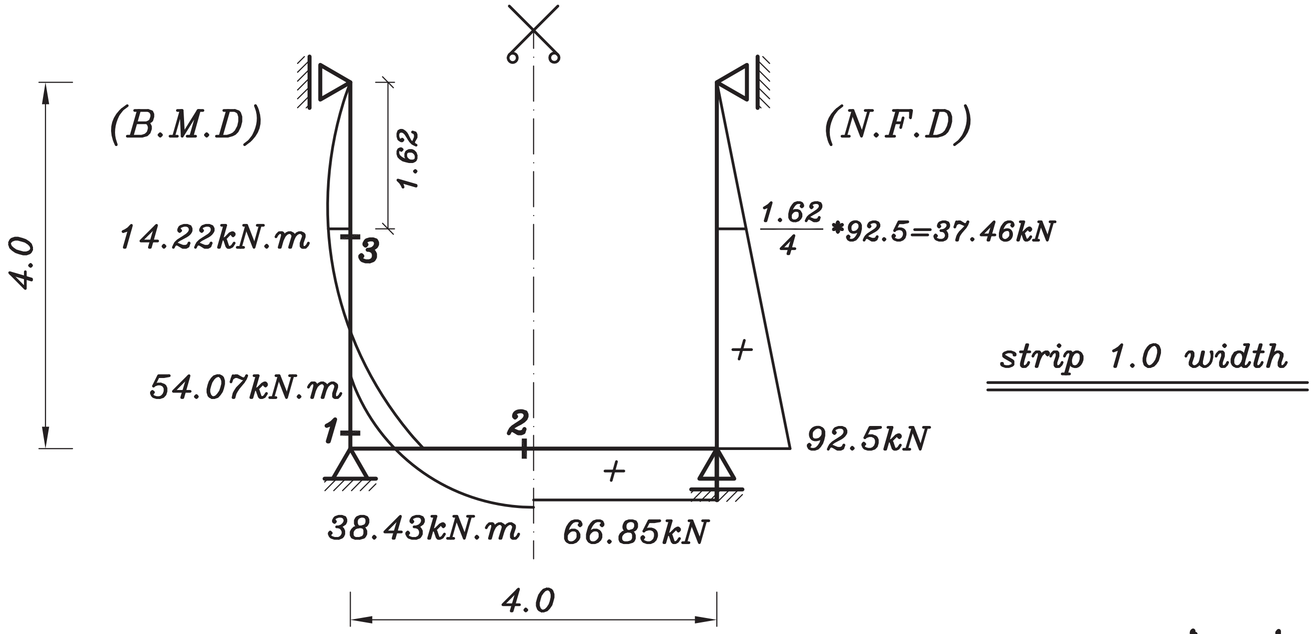
Example (7)

For the given figure, it is required to:

1– Design the critical sections.

2– Draw details of R.F.T. to scale 1:25

Given : $f_{cu}=25\ N/mm^2$, steel used is 360/520



ـ ملحوظة

فى هذا المثال نجد ان [sec(1) is a water sec.] لذلك نقوم بحساب تخانته من (stage I) ثم نحسب التسليح من (stage II)

بينما [sec(2,3) are air sec.] لذلك نفرض التخانة (t_s =250mm) ثم نحسب التسليح من (stage II) مباشرة

Sec (1–1) water section

$M_{working}=54.07\ kN.m$, $T_{working}=92.5\ kN$, $b=1000mm$

Stage (I)

$$t_{(mm)}=\sqrt{\frac{M*10^3}{factor}}+40mm=\sqrt{\frac{54.07*10^3}{0.28}}+40mm=479.44mm$$

$\Longrightarrow$ Take t =500mm

Check stresses

$$f_t = +\frac{T}{A} + \frac{M}{Z} = +\frac{92.5*10^3}{1000*500} + \frac{54.07*10^6}{1000*(500)^2/6}$$

$$= 0.19 + 1.30 = 1.49 \ N/mm^2$$

$$t_v = 500[1 + \frac{0.19}{1.30}] = 573mm \Longrightarrow f_{ct} = \frac{0.6\sqrt{25}}{1.7} = 1.76 N/mm^2$$

$$\Longrightarrow f_t < f_{ct} \quad (safe)$$

Stage (II)

$$M_{u.l.} = 1.5*54.07 = 81.11 \ kN.m \quad , \quad T_{u.l.} = 1.5*92.5 = 138.75 \ kN$$

$$e = \frac{M_{u.l.}}{T_{u.l.}} = \frac{81.11}{138.75} = 0.58m > \frac{t}{2} - cover$$

$$e_s = e - \frac{t}{2} + c = 0.58 - \frac{0.50}{2} + 0.04 = 0.37m$$

$$M_{us} = 138.75*0.37 = 51.34 kN.m$$

$$460 = C_1 \sqrt{\frac{51.34*10^6}{1000*25}} \qquad C_1 = 10.2 \quad \& \quad J = 0.826$$

$$A_s = \frac{1}{\beta_{cr}} * [\frac{M_{us}}{J*d*f_y} + \frac{T_{u.l.}}{f_y/\gamma_s}] \text{ assume } \phi 16 \text{ used } \Longrightarrow \beta_{cr} = 0.75$$

$$A_s = \frac{1}{0.75}[\frac{51.34*10^6}{0.826*460*360} + \frac{138.75*10^3}{360/1.15}]$$

$$A_s = 1091.41 mm^2/m' \Longrightarrow 6\phi 16/m'$$

Sec (2-2) air section

$$M_{working} = 38.43 \ kN.m \ , \quad T_{working} = 66.85 \ kN \quad , \quad b = 1000mm$$

Stage (II) assume t=250mm

$$M_{u.l.} = 1.5*38.43 = 57.65 \ kN.m \quad , \quad T_{u.l.} = 1.5*66.85 = 100.28 \ kN$$

$$e = \frac{M_{u.l.}}{T_{u.l.}} = \frac{57.65}{100.28} = 0.57m > \frac{t}{2} - cover$$

$$e_s = e - \frac{t}{2} + c = 0.57 - \frac{0.25}{2} + 0.04 = 0.49m$$

$$M_{us} = 100.28 * 0.49 = 49.14 kN.m$$

$$210 = C_1 \sqrt{\frac{49.14*10^6}{1000*25}} \qquad C_1 = 4.74 \quad \& \quad J = 0.82$$

$$A_s = \frac{1}{\beta_{cr}} * [\frac{M_{us}}{J*d*f_y} + \frac{T_{u.l.}}{f_y/\gamma_s}] \text{ assume } \phi 16 \text{ used} \Longrightarrow \beta_{cr} = 0.93$$

$$A_s = \frac{1}{0.93} [\frac{49.14*10^6}{0.82*210*360} + \frac{100.28*10^3}{360/1.15}]$$

$$A_s = 1196.80 mm^2/m' \Longrightarrow 6\phi 16/m'$$

Sec (3-3) air section

$M_{working} = 14.22$ kN.m , $T_{working} = 37.46$ kN , $b = 1000mm$

Stage (II) assume t=250mm

$M_{u.l.} = 1.5*14.22 = 21.33$ kN.m , $T_{u.l.} = 1.5*37.46 = 56.19$ kN

$$e = \frac{M_{u.l.}}{T_{u.l.}} = \frac{21.33}{56.19} = 0.38m > \frac{t}{2} - cover$$

$$e_s = e - \frac{t}{2} + c = 0.38 - \frac{0.25}{2} + 0.04 = 0.29m$$

$$M_{us} = 56.19 * 0.29 = 16.55 kN.m$$

$$210 = C_1 \sqrt{\frac{16.55*10^6}{1000*25}} \qquad C_1 = 8.2 \quad \& \quad J = 0.826$$

$$A_s = \frac{1}{\beta_{cr}} * [\frac{M_{us}}{J*d*f_y} + \frac{T_{u.l.}}{f_y/\gamma_s}] \text{ assume } \phi 12 \text{ used} \Longrightarrow \beta_{cr} = 1.00$$

$$A_s = \frac{1}{1.00} [\frac{16.55*10^6}{0.826*210*360} + \frac{56.19*10^3}{360/1.15}]$$

$$A_s = 444.53 mm^2/m' \Longrightarrow 5\phi 12/m'$$

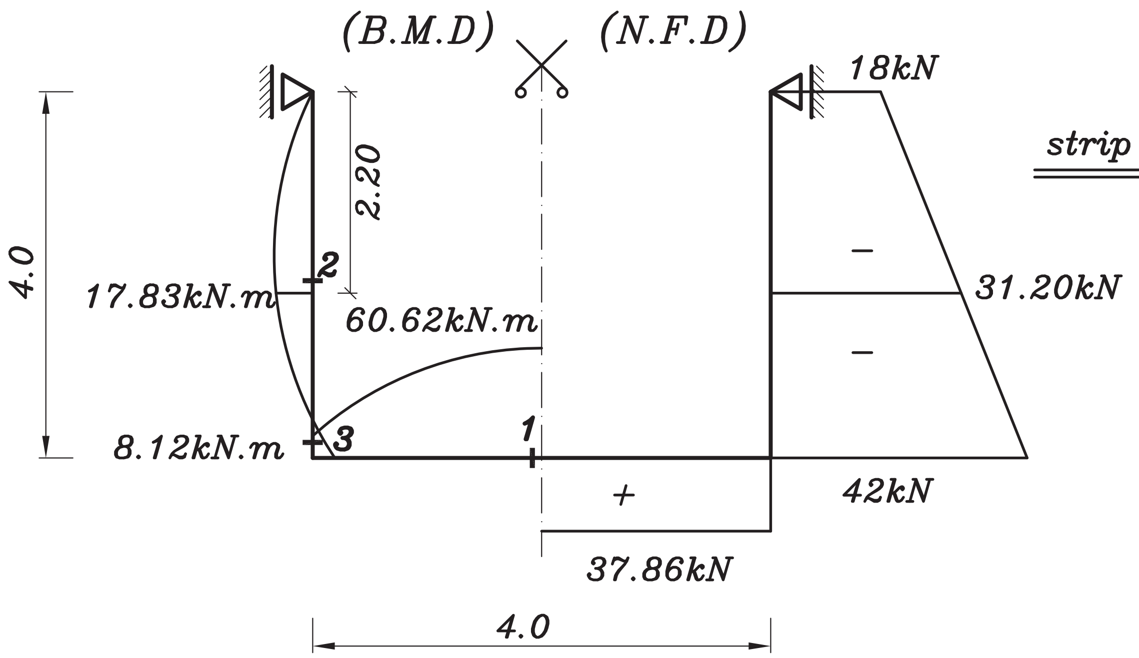
Example (8)

For the given figure, it is required to:

1– Design the critical sections.

2– Draw details of R.F.T. to scale 1:25

Given : $f_{cu}=25\ N/mm^2$, steel used is 360/520



Sec (1–1) water section

$M_{working}=60.62\ kN.m$, $T_{working}=37.86\ kN$, $b=1000mm$

Stage (I)

$$t_{(mm)}=\sqrt{\frac{M*10^3}{factor}}+40mm=\sqrt{\frac{60.62*10^3}{0.28}}+40mm=505.3mm$$

$\Longrightarrow$ Take $t=550mm$

Check stresses

$$f_t=+\frac{T}{A}+\frac{M}{Z}=+\frac{37.86*10^3}{1000*550}+\frac{60.62*10^6}{1000*(550)^2/6}$$

$$=0.07+1.20=1.27\ N/mm^2$$

$$t_v = 550[1 + \frac{0.07}{1.20}] = 582mm \Longrightarrow f_{ct} = \frac{0.6\sqrt{25}}{1.7} = 1.76 N/mm^2$$

$$\Longrightarrow f_t < f_{ct} \quad (safe)$$

Stage (II)

$$M_{u.l.} = 1.5 * 60.62 = 90.93 \ kN.m \quad , \quad T_{u.l.} = 1.5 * 37.86 = 56.79 \ kN$$

$$e = \frac{M_{u.l.}}{T_{u.l.}} = \frac{90.93}{56.79} = 1.60m > \frac{t}{2} - cover$$

$$e_s = e - \frac{t}{2} + c = 1.60 - \frac{0.55}{2} + 0.04 = 1.37m$$

$$M_{us} = 56.79 * 1.37 = 77.58 kN.m$$

$$510 = C_1 \sqrt{\frac{77.58 * 10^6}{1000 * 25}} \qquad C_1 = 9.16 \quad \& \quad J = 0.826$$

$$A_s = \frac{1}{\beta_{cr}} * [\frac{M_{us}}{J * d * f_y} + \frac{T_{u.l.}}{f_y / \gamma_s}] \text{ assume } \phi 16 \text{ used} \Longrightarrow \beta_{cr} = 0.75$$

$$A_s = \frac{1}{0.75} [\frac{77.58 * 10^6}{0.826 * 510 * 360} + \frac{56.79 * 10^3}{360 / 1.15}]$$

$$A_s = 923.96 mm^2 / m' \Longrightarrow 5\phi 16 / m'$$

لاحظ ان تخانة $(t_{sec1} = 550mm)$ سوف تستمر على طول ارضية الخزان لانه خزان $(rested)$ حتى تكون ارضية الخزان $(rigid)$

Sec (2-2) air section

$$M_{working} = 17.83 \ kN.m \ , \quad N_{working} = 31.20 \ kN \quad , \quad b = 1000mm$$

Stage (II) assume t=250mm

$$M_{u.l.} = 1.5 * 17.83 = 26.75 \ kN.m \quad , \quad N_{u.l.} = 1.5 * 31.20 = 46.80 \ kN$$

$$e = \frac{M_{u.l.}}{T_{u.l.}} = \frac{26.75}{46.80} = 0.57m \Longrightarrow \frac{e}{t} = \frac{0.57}{0.25} = 2.28 > 0.5 \ (big \ ecc.)$$

$$e_s = e + \frac{t}{2} - c = 0.57 + \frac{0.25}{2} - 0.04 = 0.66m$$

$M_{us}=46.80*0.66=30.73 kN.m$

$$210=C_1\sqrt{\frac{30.73*10^6}{1000*25}} \qquad C_1=5.99 \quad \& \quad J=0.826$$

$$A_s=\frac{1}{\beta_{cr}}*\left[\frac{M_{us}}{J*d*f_y}-\frac{N_{u.l.}}{f_y/\gamma_s}\right] \text{ assume } \phi 12 \text{ used} \Longrightarrow \beta_{cr}=1.00$$

$$A_s=\frac{1}{1.00}\left[\frac{30.73*10^6}{0.826*210*360}-\frac{46.80*10^3}{360/1.15}\right]$$

$$A_s=342.61 mm^2/m' \Longrightarrow 5\phi 12/m'$$

Sec (3-3) water section

$M_{working}=8.12$ kN.m , $N_{working}=42$ kN , $b=1000mm$

Stage (I)

$$t_{(mm)}=\sqrt{\frac{M*10^3}{factor}}-20mm=\sqrt{\frac{8.12*10^3}{0.38}}-20mm=126.2mm$$

$\Longrightarrow$ Take $t=250mm$

Check stresses

$$f_t=-\frac{N}{A}+\frac{M}{Z}=-\frac{42.0*10^3}{1000*250}+\frac{8.12*10^6}{1000*(250)^2/6}$$

$$=-0.17+0.78=0.61 \ N/mm^2 < f_{ct} \ (safe)$$

Stage (II)

$M_{u.l.}=1.5*8.12=12.18$ kN.m , $N_{u.l.}=1.5*42.0=63.0$ kN

$A_s=5\phi 12/m'$

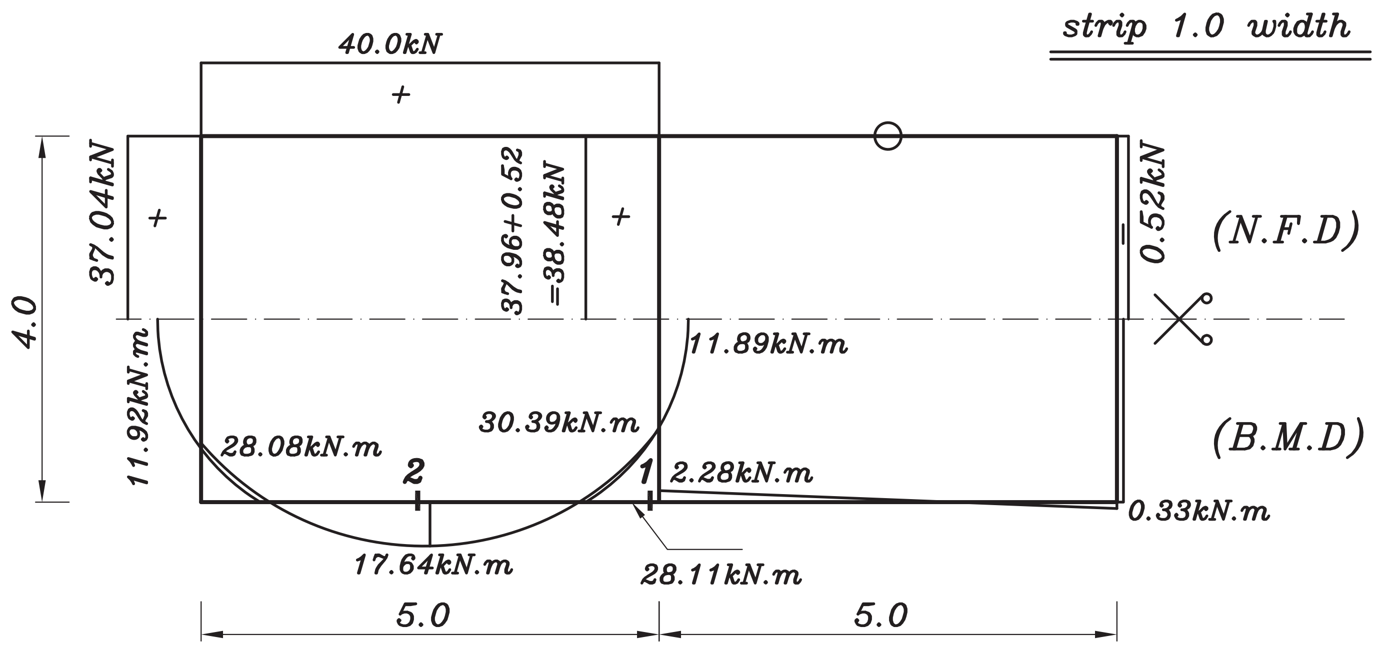
Example (9)

For the given figure, it is required to:

1– Design the critical sections.

2– Draw details of R.F.T. to scale 1:25

Given : $f_{cu}=25\ N/mm^2$, steel used is 360/520



Sec (1–1) water section

$M_{working}=30.39\ kN.m$, $T_{working}=40.0\ kN$, $b=1000mm$

Stage (I)

$$t_{(mm)}=\sqrt{\frac{M*10^3}{factor}}+40mm=\sqrt{\frac{30.39*10^3}{0.30}}+40mm=358.28mm$$

⟹ Take $t=400mm$

Check stresses

$$f_t=+\frac{T}{A}+\frac{M}{Z}=+\frac{40.0*10^3}{1000*400}+\frac{30.39*10^6}{1000*(400)^2/6}$$

$$=0.10+1.14=1.24\ N/mm^2<f_{ct}\quad (safe)$$

Stage (II)

$M_{u.l.}=1.5*30.39=45.59 \ kN.m \quad , \quad T_{u.l.}=1.5*40.0=60.0 \ kN$

$e=\frac{M_{u.l.}}{T_{u.l.}}=\frac{45.59}{60.0}=0.76m > \frac{t}{2} - cover$

$e_s = e-\frac{t}{2}+c=0.76-\frac{0.40}{2}+0.04=0.60m$

$M_{us}=60.0*0.60=36.00kN.m$

$360=C_1 \sqrt{\frac{36.00*10^6}{1000*25}} \qquad C_1=9.49 \quad \& \quad J=0.826$

$A_s=\frac{1}{\beta_{cr}}*[\frac{M_{us}}{J*d*f_y}+\frac{T_{u.l.}}{f_y/\gamma_s}]$ assume $\phi 12$ used $\Longrightarrow \beta_{cr}=0.85$

$A_s=\frac{1}{0.85}[\frac{36.00*10^6}{0.826*360*360}+\frac{60.0*10^3}{360/1.15}]$

$A_s=621.13mm^2/m' \Longrightarrow 6\phi 12/m'$

Sec (2-2) air section

$M_{working}=17.64 \ kN.m \ , \quad T_{working}=40.0 \ kN \quad , \quad b=1000mm$

Stage (II) assume t=250mm

$M_{u.l.}=1.5*17.64=26.46 \ kN.m \quad , \quad T_{u.l.}=1.5*40.0=60.0 \ kN$

$e=\frac{M_{u.l.}}{T_{u.l.}}=\frac{26.46}{60.0}=0.44m > \frac{t}{2} - cover$

$e_s = e-\frac{t}{2}+c=0.44-\frac{0.25}{2}+0.04=0.36m$

$M_{us}=60.0*0.36=21.36kN.m$

$210=C_1 \sqrt{\frac{21.36*10^6}{1000*25}} \qquad C_1=7.18 \quad \& \quad J=0.826$

$A_s=\frac{1}{\beta_{cr}}*[\frac{M_{us}}{J*d*f_y}+\frac{T_{u.l.}}{f_y/\gamma_s}]$ assume $\phi 12$ used $\Longrightarrow \beta_{cr}=1.00$

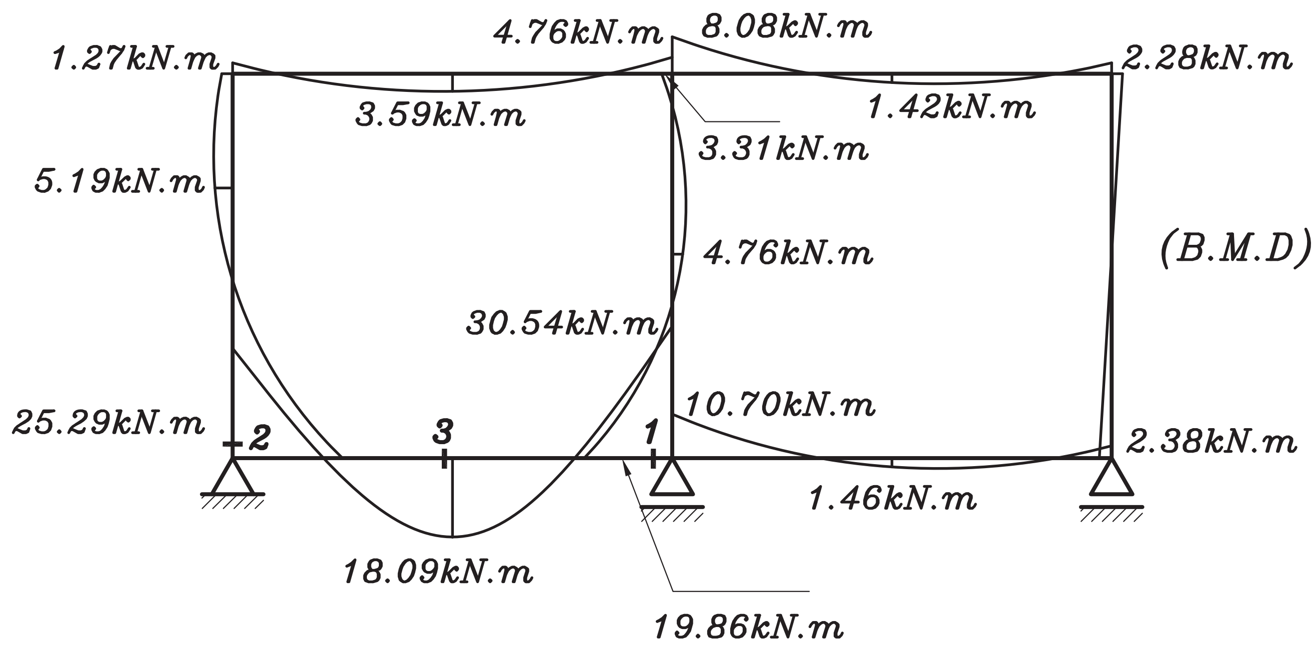
Example (10)

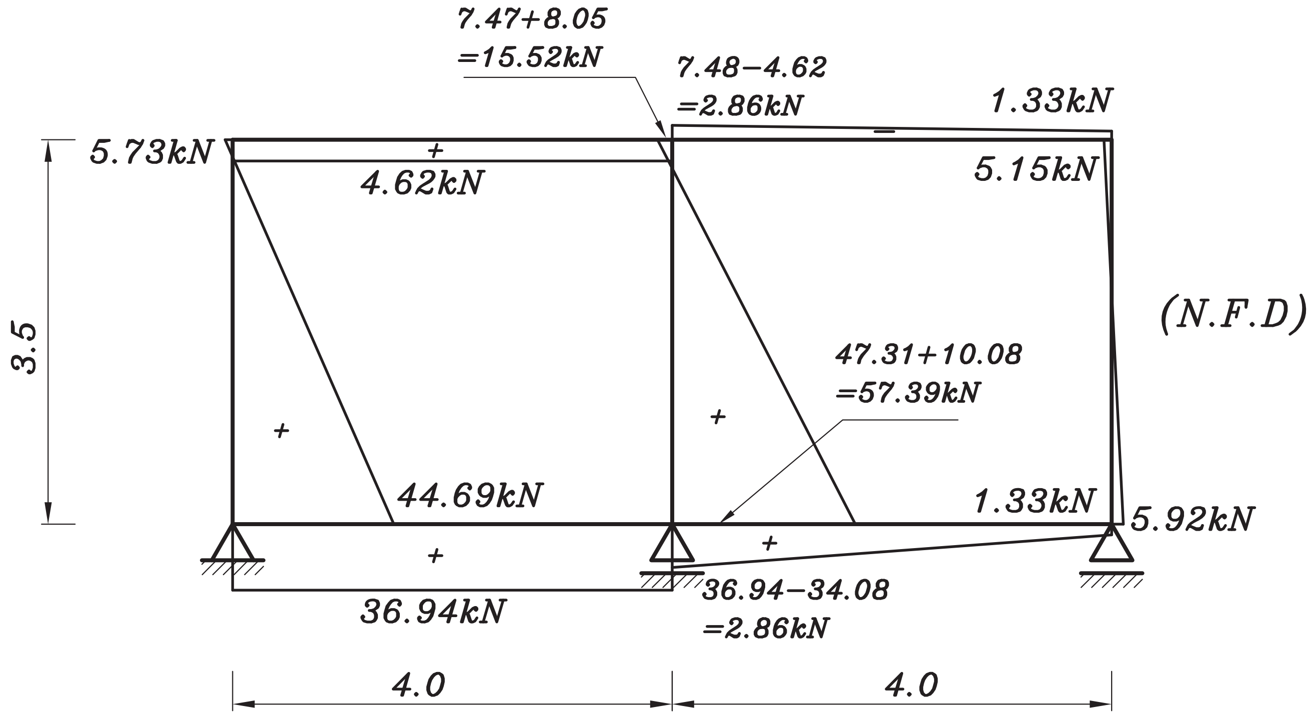
For the given figure, it is required to:

1- Design the critical sections.

2- Draw details of R.F.T. to scale 1:25

Given : f_{cu}=25 N/mm^2 , steel used is 360/520





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Sec (1-1) water section

$M_{working}=30.54$ kN.m , $T_{working}=44.69$ kN , b=1000mm

Stage (I)

$$t_{(mm)}=\sqrt{\frac{M*10^3}{factor}}+40mm=\sqrt{\frac{30.54*10^3}{0.30}}+40mm=359.06mm$$

$\Longrightarrow$ Take t =400mm

Check stresses

$$f_t=+\frac{T}{A}+\frac{M}{Z}=+\frac{44.69*10^3}{1000*400}+\frac{30.54*10^6}{1000*(400)^2/6}$$

$$=0.11+1.15=1.26\ N/mm^2 < f_{ct} \quad (safe)$$

Stage (II)

$M_{u.l.}=1.5*30.54=45.81$ kN.m , $T_{u.l.}=1.5*44.69=67.04$ kN

$$e=\frac{M_{u.l.}}{T_{u.l.}}=\frac{45.81}{67.04}=0.68m>\frac{t}{2}-cover$$

$$e_s=e-\frac{t}{2}+c=0.68-\frac{0.40}{2}+0.04=0.52m$$

$$M_{us}=67.04*0.52=35.08kN.m$$

$$360=C_1\sqrt{\frac{35.08*10^6}{1000*25}} \qquad C_1=9.61 \quad \& \quad J=0.826$$

$$A_s=\frac{1}{\beta_{cr}}*[\frac{M_{us}}{J*d*f_y}+\frac{T_{u.l.}}{f_y/\gamma_s}] \text{ assume } \phi 12 \text{ used} \Longrightarrow \beta_{cr}=0.85$$

$$A_s=\frac{1}{0.85}[\frac{35.08*10^6}{0.826*360*360}+\frac{67.04*10^3}{360/1.15}]$$

$A_s=670.45mm^2/m'$ $\Longrightarrow$ $6\phi 12/m'$

Sec (2–2) water section

$M_{working}=25.29\ kN.m$, $T_{working}=44.69\ kN$, $b=1000mm$

Stage (I)

$$t_{(mm)}=\sqrt{\frac{M*10^3}{factor}}+40mm=\sqrt{\frac{25.29*10^3}{0.30}}+40mm=330.34mm$$

$\Longrightarrow$ Take $t=350mm$

Check stresses

$$f_t=+\frac{T}{A}+\frac{M}{Z}=+\frac{44.69*10^3}{1000*350}+\frac{25.29*10^6}{1000*(350)^2/6}$$

$$=0.13+1.24=1.37\ N/mm^2<f_{ct}\quad (safe)$$

Stage (II)

$M_{u.l.}=1.5*25.29=37.94\ kN.m$, $T_{u.l.}=1.5*44.69=67.04\ kN$

$A_s=6\phi 12/m'$

Sec (3–3) air section

$M_{working}=18.09\ kN.m$, $T_{working}=36.94\ kN$, $b=1000mm$

Stage (II) assume $t=\underline{250mm}$

$M_{u.l.}=1.5*18.09=27.14\ kN.m$, $T_{u.l.}=1.5*36.94=55.41\ kN$

$A_s=5\phi 12/m'$