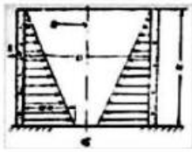
Lecture 12

Continued from lecture 11

CYLNDRICAL TANKS WITH FIXED BASE, FREE TOP

Coefficients for Tension in Circular Rings
Triangular Load
T = Coefficient x wHR kg. per m.
Positive sign indicates tension.



$\frac{H^2}{D_t}$	Coefficient at Point									
	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H
0.4	+0.149	+0.134	+0.120	+0.101	+0.062	+0.066	+0.049	+0.029	+0.014	+0.004
0.2	+0.263	+0.239	+0.215	+0.190	+0.160	+0.130	+0.096	+0.063	+0.034	+0.010
1.2	+0.283	+0.271	+0.254	+0.234	+0.209	+0.180	+0.142	+0.099	+0.054	+0.016
1.6	0.265	+0.268	+0.268	+0.266	+0.250	+0.226	+0.185	+0.134	+0.075	+0.023
2.0	+0.234	+0.251	+0.273	+0.285	+0.285	+0.274	+0.232	+0.172	+0.104	+0.031
3.0	+0.134	+0.203	+0.267	+0.322	+0.357	+0.362	+30.30	+0.262	+0.157	+0.052
4.0	+0.067	+0.164	+0.256	+0.389	+0.403	+0.429	+0.409	+0.334	+0.210	+0.073
5.0	+0.025	+0.137	+0.245	+0.346	+0.428	+0.477	+0.469	+0.398	+0.259	+0.092
6.0	+0.018	+0.119	+0.234	+0.344	+0.441	+0.505	+0.514	+0.447	+0.301	+0.112
8.0	-0.011	+0.104	+0.218	+0.335	+0.443	+0.534	+0.575	+530	+0.381	+0.151
10.0	-0.011	+0.098	+0.028	+0.323	+0.437	+0.542	+0.608	+0.589	+0.440	+0.179
12.0	-0.005	+0.097	+0.202	+0.312	+0.429	+0.543	+0.628	+0.633	+0.494	+0.211
14.0	-0.002	+0.098	+0.200	+0.306	+0.420	+0.539	+0.639	+0.666	+0.541	+0.241
16.0	0.000	+0.099	+0.199	+0.300	+0.413	+0.531	+0.541	+0.687	+0.582	+0.265

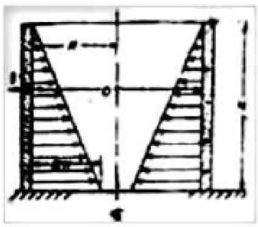
Coefficient of Point

	0.75H	0.80H	0.85H	0.90H	0.5H
20	+0.716	+0.654	+0.520	+0.325	+0.115
24	+0.746	+0.702	+0.577	+0.372	+0.137
32	+0.782	+0.768	+0.663	+0.459	+0.182
40	+0.800	+0.805	+0.731	+0.530	+0.217
48	+0.701	+0.828	+0.785	+0.593	+0.254
56	+0.763	+0.838	+0.824	+0.536	+0.285

Table 2.A

Table 2.B
CYLINDRICAL TANKS

Coefficients for Moments in Cylindrical Walls.
Triangular Load
Moments = Coefficient x wH^3 kg .m per m.
Positive sign indicates tension in the outside



$\frac{H^2}{D_t}$	Coefficient at Point									
	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	1.0H
0.4	+.0005	+.0014	+.0021	.0007	-.0063	-.0150	-.0302	-.0529	-.0616	-.1205
0.2	+.0011	+.0037	+.0065	+.0060	+.0070	+.0023	-.0065	-.0234	-.0445	-.0795
1.2	+.0013	+.0043	+.0077	+.0103	.0113	+.0090	+.0022	-.0108	-.0311	-.0005
1.6	+.0011	+.0041	+.0075	+.0107	+.0131	+.0111	+.0058	-.0051	-.0222	-.0505
2.0	+.0010	+.0035	+.0065	+0089	+.0120	+.0115	+.0075	-.0021	-.0135	-.0436
3.0	+.0006	+.0024	+.0047	+.0071	+.0090	+.0097	+.0077	+.0012	-.0119	-.0333
4.0	+.0002	+.0015	+.0028	+.0067	+.0065	+.0077	+.0069	+.0025	-.0080	-.0266
5.0	+.0002	+.0006	+.0016	+.0029	+.0046	+.0059	+.0059	+.0028	-.0058	-.0222
6.0	+.0001	+.0008	+.0008	+.0019	+.0032	+.0046	+.0051	+.0029	-.0041	-.0187
8.0	.0000	+.0001	+.0008	+.0008	+.0016	+.0028	+.0038	+.0029	-.0022	-.0146
10.0	.0000	.0000	+.0001	+.0004	+.0007	+.0019	+.0029	+.0025	-.0002	-.0122
12.0	.0000	-.0001	+.0001	+.0002	+.0008	+.0013	+.0023	+.0026	-.0006	-.0104
14.0	.0000	.0000	.0000	.0000	+.0001	+.0009	+.0019	+.0023	-.0001	-.0090
16.0	.0000	.0000	-.0001	-.0001	-.0001	+.0004	+.0013	+.0019	-.0001	-.0079

Coefficient of Point

	0.80H	0.85H	0.90H	0.25H	1.00H
20	+.0016	+.0014	+.0005	-.0018	-.0003
24	+.0012	+.0012	+.0007	-.0013	-.0053
32	+.0007	+.0009	+.0007	-.0008	-.0040
40	+.0002	+.0005	+.0006	-.0005	-.0032
48	+.0000	+.0001	+.0006	-.0003	-.0026
56	+.0000	+.0000	+.0006	-.0001	-.0023

2. Circular tanks hinged at base and free at top.

At the top shear force and bending moment will be zero. At the base deflection and *B.M.* will be zero.

$$At\,x = H, \quad -M = EI\frac{d^2y}{dx^2} = 0$$

$$At\,x = H, \quad -F = EI\frac{d^3y}{dx^3} = 0$$

$$At\,x = 0, \quad EIy = 0$$

$$At\,x = 0, \quad -M = EI\frac{d^2v}{dx^2} = 0$$

Thus four equations are obtained. These four equations can be solved for four constant. Hence values of *Pc* and *Pr* can be found.

Table 3C gives coefficients for ring tension and *B.M* at various heights and shear at the base.

Table 3. A

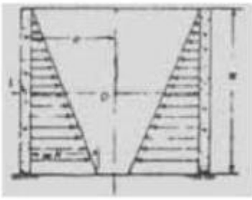
CYLINDRICAL TANKS WITH HINGED BASE AND FREE TOP

Coefficients for Tension in Circular Rings

Triangular Load

T = Coefficient x *wHR* kg. per m.

Positive sign indicates tension.



H^2	Coefficient at Point									
D_t	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H
0.4	+0.474	+0.440	+0.395	+0.352	+0.308	+0.264	+0.215	+0.166	+0.111	+0.007
0.2	+0.423	+0.402	+0.381	+0.358	+0.330	+0.297	+0.249	+0.202	+0.145	+0.076
1.2	+0.350	+0.355	+0.361	+0.362	+0.358	+0.343	+0.309	+0.255	+0.185	+0.098
1.6	+0.271	+0.302	+0.341	+0.369	+0.385	+0.385	+0.362	+0.314	+0.233	+0.124
2.0	+0.205	+0.260	+0.321	+0.373	+0.411	+0.434	+0.419	+0.369	+0.280	+0.151
3.0	+0.074	+0.179	+0.281	+0.375	+0.449	+0.506	+0.519	+0.479	+0.875	+0.210
4.0	-0.017	+0.137	+0.253	+0.267	+0.469	+0.545	+0.579	+0.553	+0.447	+0.256
5.0	-0.006	+0.114	+0.235	+0.356	+0.469	+0.562	+0.617	+0.606	+0.503	+0.294
6.0	-0.011	+0.103	+0.223	+0.343	+0.463	+0.566	+0.639	+0.643	+0.547	+0.327
8.0	-0.015	+0.096	+0.208	+0.324	+0.443	+0.564	+0.661	+0.697	+0.621	+0.386
10.0	-0.006	+0.095	+0.200	+0.311	+0.423	+0.552	+0.666	+0.730	+0.676	+0.433
12.0	-0.002	+0.097	+0.197	+0.302	+0.417	+0.541	+0.664	+0.750	+0.720	+0.477
14.0	0.000	+0.000	+0.197	+0.299	+0.408	+0.531	+0.659	+0.761	+0.752	+0.513
16.0	+0.202	+0.100	+0.198	+0.299	+0.403	+0.521	+0.650	+0.764	+0.776	+0.543

Coefficient of Point

	0.75H	0.80H	0.85H	0.90H	0.95H
20	+0.812	+0.817	+0.756	+0.603	+0.0344
24	+0.816	+0.839	+0.793	+0.647	+0.0377
32	+0.814	+0.861	+0.847	+0.721	+0.436
40	+0.802	+0.866	+0.880	+0.778	+0.483
48	+0.791	+0.864	+0.900	+0.820	+0.527
56	+0.781	+0.859	+0.911	+0.852	+0.563

3. Cylindrical tank with top slab and fixed base

At top deflection will be zero and slope will be as for the slope of top slab. These conditions will give two equations. At bottom, slope and deflection will be zero. Thus four equations are obtained which can be solved for constants C_1, C_2, C_3 and C_4. Hence values of P_0 and P_r can be calculated. *B. M.* and ring tension at various heights can be calculated.

4. Cylindrical tanks with Domes at top and bottom.

The slopes and deflections of the wall at top and bottom will be equal to slopes and deflections for the domes at top and bottom respectively. Thus four equations can be formed which can be solved for four constants. Knowing equation of elastic curve, values of P_0 and P_r can be calculated.

Ex. **2.** Design the tank of Problem 17° 1 if the tank is fixed at the base and free at the top.

Sol. $D = 13$ m., $H = 4$ m.

Assume thickness of wall = 15 cm.

$$\frac{H^2}{Dt} = \frac{4^2}{13 \times 0.15} = 8.20$$

Coefficients for hoop tension and *B.M* for various heights are found by interpolation from Table 17.2. These coefficients are given in Table 17.4.

Maximum hoop tension occurs at 0.6 *H* from top

Maximum hoop tension

$$= 0.5783 \text{ x } w\frac{HD}{2}$$

$$= 0.5783 \text{ x } 1000 \text{ x } \frac{4 \times 13}{2}$$

$$= 14{,}736 \text{ kg}$$

TABLE 4

Depth	Coefficient for hoop tension	Depth	Coefficient for *B.M.*
0°0*H*	- 0°011	0°1*H*	- 0°0000
0°1*H*	+ 0°1034	0°2*H*	+ 0°00009
0°2*H*	+ 0°2170	0°3*H*	+ 0°00019
0°3*H*	+ 0°3338	0°4*H*	+ 0°00076
0°4*H*	+ 0°4424	0°5*H*	+ 0°00151
0°5*H*	+ 0°5348	0°6*H*	+ 0°00271
0°6*H*	+ 0°5783	0°7*H*	+ 0°00371
0°7*H*	+ 0°5359	0°8*H*	+ 0°00289
0°8*H*	+ 0°3869	0°9*H*	- 0°0021
0°9*H*	+ 0°1538	1*H*	- 0°01436
H	-		

Steel area required = $\frac{14,736}{1000}$ = 14°736 cm^2.

Provide 12 mm ϕ bars at 14 cm. centers on both faces.

A_l = 16°16 cm^2

The reinforcement is provided for 0.8*H* to 0.3*H* from top. In the remaining portion provide 12 mm ϕ bars at 20 cm centers.

Tensile stress in concrete

$$= \frac{14,736}{100 \times 15 + (m-11) \times 16.1}$$

$$= \frac{14,736}{1500 + 12 \times 16.16}$$

$$= \frac{14,736}{1693.92} = 8.7\, kg/cm^2.$$

Safe

Maximum position B.M. (tension outside) occurs at 0.7*H* from top. Maximum +ve *B.M.* per metre height

= coefficient x wH^3

= 0.0037 x 1000 x 4^3

= + 237.4 kg. m.

Maximum – ve *B.M.* (tension inside)

= 0.01436 x 1000 x 4^3

= -919.1 kg. m.

Effective depth of 15 – 4 = 11 cm. is provided

Area of steel required for +ve *B.M.* on outer face

$$= \frac{23,740}{0.84 \times 11 \times 1000} = 2.57 \text{ cm}^2$$

Minimum percentage of steel to be provided

$$= 0.3 - \frac{0.1 \times 5}{35}$$

$$= 0.3 - 0.014 = 0.286.$$

Minimum steel area required

$$= \frac{0.286}{100} \text{ x } 15 \text{ x } 100 = 4.290 \text{ cm}^2$$

Minimum reinforcement on one face = 2.145 cm^2

Provide 8 mm ϕ bars at 20cm. centers.

Area of steel required for – ve *B.M.* on inner face

$$= \frac{91{,}910}{0.84 \times 11 \times 1000} = 9.65 \text{ cm}^2$$

Provide 12 mm bars at 11 cm centers.

Shear force

Maximum shear at base of wall

$$= 0.1724 \text{ w}H^2$$

$$= 0.1724 \text{ x } 1000 \text{ x } 4 \text{ x } 4 = 2758.4 \text{ kg}$$

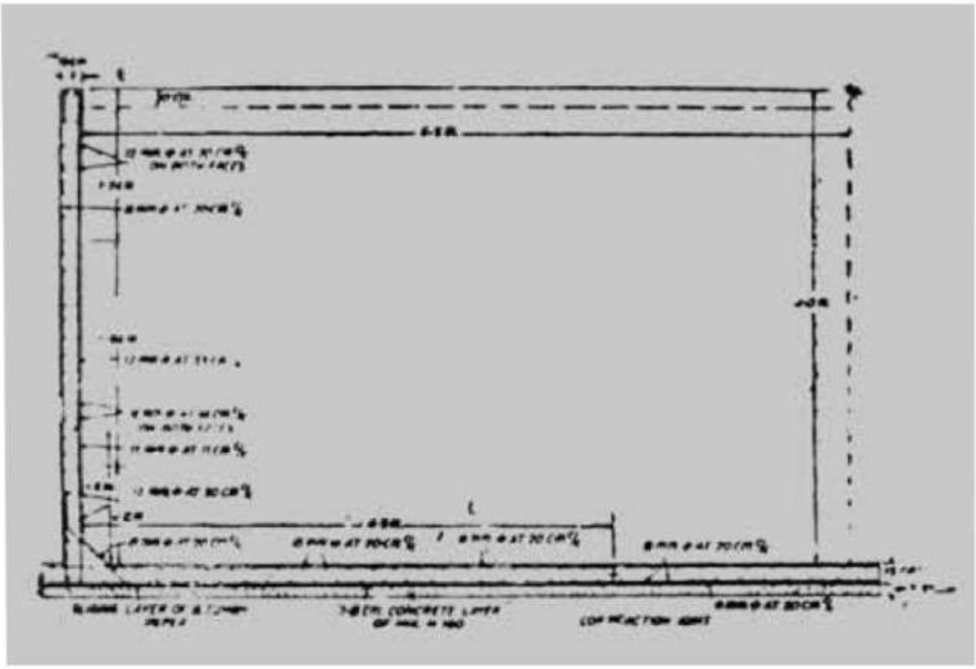


Fig. 7

Shear stress $$= \frac{2758.4}{0.84 \times 11 \times 100} = 2.99 \text{ kg / cm}^2$$

Safe.

Check for Bond

$$\text{Bond stress} = \frac{2758.4}{0.84 \times 11 \times \frac{100}{11} \times 3.76}$$

$$= 8.73 \text{ kg/ cm}^2$$

Safe.

Base. Provide 15 cm thick base slab with 8 mm φ bars at 20 cm centers both ways at top and bottom.

8. Approximate method of design of circular tanks with fixed base. In the approximate method of design of circular tanks it is assumed that some portion of the tank at base acts as cantilever and thus some load at bottom is taken by the cantilever effect. Load in the top portion is taken by the hoop tension caused in the top portion. The cantilever effect will depend on the dimensions of the tank and thickness of the wall for between 6 to 12, the cantilever portion may be assumed at H/3 or 1 m from base whichever is more. For H^2/D_t between 12 to 30, the cantilever portion may be assumed as H/4 or 1 m from base, whichever is more.

In Fig. 8 *AB* is the height of tank and *ABC* pressure diagram. *ADB* is taken as pressure causing hoop tension and *DBC* is taken as cantilever load. The maximum *B.M.* occurs at the base.

The steel for hoop tension is provided on both faces. For the bottom portion BD reinforcement for hoop tension is provided in addition to steel required for bending.

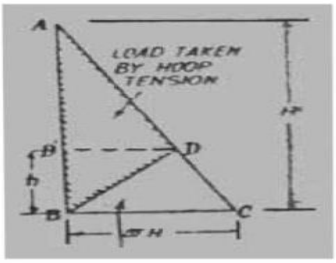


Fig.8

Ex. 3 Design the water tank of problem 1 by approximate method.

Sol. $D = 13$ m $\qquad H = 4$ m

Assume thickness of wall $t = 15$ cm

$$\frac{H^2}{Dt} = \frac{4 \times 4}{13 \times 0.15} = 8.276$$

It is assumed that bottom $\frac{H}{3}$ i.e. $\frac{4}{3}$ m acts as cantilever.

$$\text{Maximum hoop tension} = \frac{pD}{2}$$

$$= \frac{8000}{3} \times \frac{13}{2}$$

= 17,333 kg. per meter height.

Area of steel required

$$= \frac{17{,}333}{1000} = 17.33 \text{ cm}^2$$

Provide 12 mm φ bars at 13 cm centers on both faces

$$\text{Maximum } B.M. = \frac{1}{2} \times 4000 \times \frac{4}{3} \times \frac{1}{3} \times \frac{4}{3}$$

$$= \frac{32{,}000}{27} = 1185 \, kg.m$$

Effective depth = 15-4 = 11 cm

Area of steel required

$$= \frac{1185 \times 100}{0.84 \times 11 \times 1000} = 12.84 cm^3$$

Provide 12 mm ϕ bars at 8 cm center.

Distribution steel

$$\% \text{ reinforcement} = \frac{0.3 - 0.1 \times (15 - 10)}{(45 - 10)} = 0.286$$

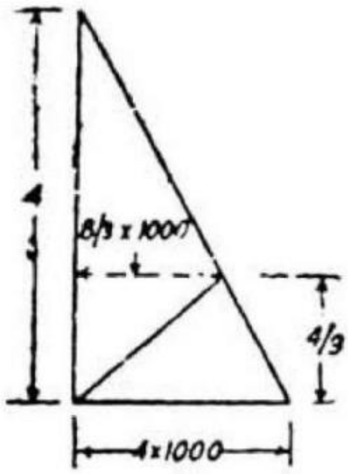


Fig. 9

$$\text{Steel area} = \frac{0.286}{100} \times 15 \times 100$$
$$= 4.29 \text{ cm}^2$$

Provide 8 mm ϕ bars at 22 cm. centers on each face.

9. **Rectangular tank**. Rectangular tanks are provided when small capacity tanks are required. For small capacities circular tanks prove uneconomical as the formwork for circular tanks is very costly. The rectangular tanks should be preferably square in plan from point of view of economy. It is desirable that longer side should not be greater than twice the smaller side.

In rectangular tanks moments are caused in two directions. The exact analysis is rather difficult and such tanks are designed by approximate methods.

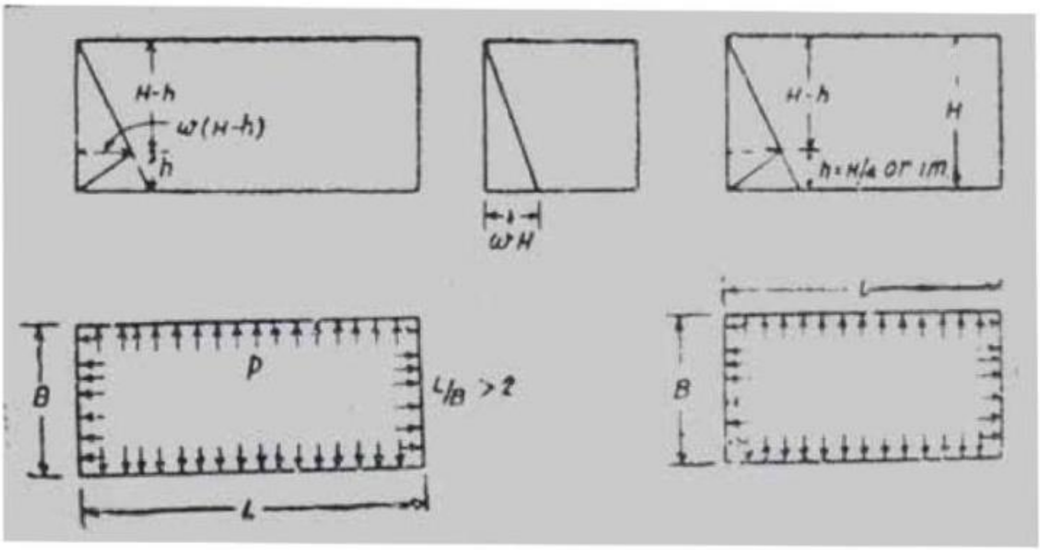
For rectangular tanks in which ratio of length to breadth is less than 2, tank walls are designed as continuous frame subjected to pressure varying from zero at to top to maximum at H/4 or 1 m., from base, whichever is more. The bottom portion H/4 (or) 1 m whichever is more is designed as cantilever. In addition to bending, walls are subjected to direct tension caused by the hydrostatic pressure on the walls. The section is to be designed for direct tension and bending. Bending moments in the walls are found by moment distribution. Direct tension in long walls

$$= \frac{w(H - h) \times B}{2}$$

and direct tension in short walls

$$= \frac{w(H-h) \times L}{2}$$

For rectangular tanks in which ratio of length to breadth is greater than 2, the long walls are designed as cantilevers and short



(b) **Fig. 10** **(a)**

walls as slabs supported on long walls. Bottom portion of short walls H/4 or 1 m whichever is more, is designed as cantilever.

Maximum *B.M.* in long walls at base

$= \frac{1}{2}$ w H x H x $\frac{H}{3}$ $= \frac{wH^3}{6}$ In the short walls maximum *B.M.* occurs at support and is given by $\frac{w(H-h)B^2}{12}$ *B.M.* at center of short walls is taken as $\frac{w(H-h)B^2}{16}$. For bottom portion of short wall, which is designed as cantilever maximum *B.M.* is given by $\frac{wH^3}{6}$ or $\frac{wH \times 1}{6}$ whichever is greater. In addition to *B.M.* short walls and long walls are subjected to direct tension. Direct tension on long walls is given by $\frac{w(H-h) \times B}{2}$.For short walls it is assumed that end one meter width of long wall contributes to direct tension on the short walls. Direct tension on short wall is $w(H\text{-}h)$.

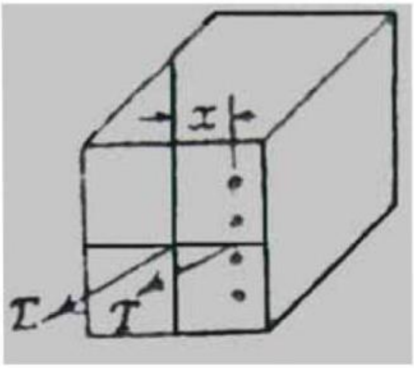


Fig. 10 (c)

Design of section for tension. It is assumed that entire tension is taken by steel. Let T be tension. Net $B.M. = M - T \times x$. Steel reinforcement is provided for $B.M.$ of $M - T \times x$ and direct tension T as shown in Fig. 10 (c).

Ex. 4 Design a rectangular tank for a capacity of 80,000 liters.

Sol. Provide height of 3.5 m for tank with free board of 15 cm.

Effective height = 335 cm.

Volume of tank to be required

$$= 80{,}000 \text{ liters} = 80{,}000{,}000 \text{ c.c}$$

Area of tank to be provided

$$= \frac{80.000{,}000}{335} = 238{,}900 \text{ cm.}^2$$

Provide length of 600 cm. and breadth of 400 cm.

$$\frac{L}{B} = \quad = 1.5 < 2$$

The wall of tanks are to be designed as continuous slab.

$$\frac{H}{4} = \frac{3.50}{4} = 0.875 \text{ m}$$

Bottom 1 m. of tank will be designed as cantilever.

Pressure at depth of 2.5 m.

$$p = wh = 2.5 \times 1000$$
$$= 2{,}500 \text{ kg/m}^2$$

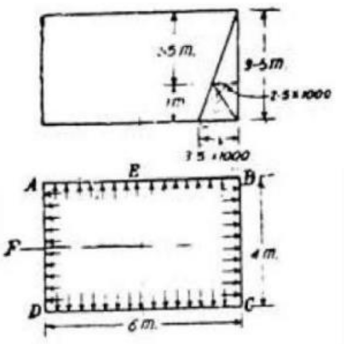


Fig. 11

Moments in the walls are found by moment distribution. As the frame is symmetrical about both axes moment distribution is done for one quarter of tank only.

Joint	A	
Member	AB	AD
Distribution Factors	0.4	0.6
Fixed Moment	-3p	+ 4/3 p
Balancing	+ 2/3 p	+ p
Final	- 7/3 P	+ 7/3p

Moment at support = $\frac{7}{3}\ p = \frac{7}{3}$ x 2500 = 5833 kg. m.

B.M. at center of long span

$= \frac{2500 \times 6^2}{8}$ - 5833 = 11,250 - 5833

= 5417 kg. m.

B.M. at center of shorter span

$= \frac{2500 \times 4^2}{8}$ - 5833

= 5000 – 5833 = - 833 kg. m.

Maximum *B.M.* = 5833 kg. m

Effective depth required

$= \sqrt{\frac{5833 \times 100}{14.11 \times 100}}$ = 20.32 cm.

Provide overall depth of 25 cm. with effective depth of 21.5 cm.

Direct tension in long wall

$= \frac{2500 \times 4}{2}$ = 5000 kg.

Direct tension in the short wall

$= \frac{2500 \times 6}{2}$ = 7500 kg.

Design of section

$c = 70$, $m = 13$, $t = 1000$ kg/cm^2 on water face

$$k = \frac{1}{1 + t/cm} = \frac{1}{1 + \frac{10.0}{70 \times 13}}$$

$$= \frac{1}{2.097} = 0.48$$

$$j = 1 - \frac{k}{3} = 1\text{-}0.16 = 0.84$$

$Q =$ ½ ckj

= ½ x 70 x 0.48 x 0.84 = 14.11

Considering effect of bending only, effective depth required

$= \sqrt{\frac{5833 \times 100}{14.11 \times 100}}$ = 20.32 cm.

Provide overall depth of 25 cm. with effective depth 21.5 cm.

Net moment = $M - T \times x$

Area of steel

$$= \frac{M - T \times x}{0.84 d \times 1000} + \frac{T}{1000}$$

$$= \frac{5833 \times 100 - 5000(21.5 - 12.5)}{0.84 \times 215 \times 1000} + \frac{5000}{1000}$$

$= 29.77 + 50 = 34.77 \text{ cm}^2$.

Provide 20 mm. ϕ bars at 8 cm. centers. Area of steel provided = 39.28 cm^2.

Overhead Tanks These tanks may be rectangular or circular. The tanks are supported on staging which consists of masonry tower or a number of columns braced together. The tank walls are designed in the same way as the walls of tanks resting on the ground. The base slab of circular tanks is designed as circular slab supported on masonry or circular beam at the end. The slab of rectangular tanks is designed as two-way slab if length is less than twice the breadth the slab is designed as one-way slab. The base slab is subjected to bending moment at the end to direct tension, caused by the water pressure acting on vertical walls.

For large tanks base slab is supported on series of beams supported on columns.

The staging consists of a number of columns braced together at intervals. The columns are assumed to be fixed at the braces as well as to elevated tank, therefore, effective length of column is taken as distance between bracings.

The wind force acting on the tank and staging produces tension on the windward side columns and compression on the leeward side columns. The force in any column is proportional to its distance from C.G. of the column group.

Let 'P' be the total wind force acting at height 'h' from the base and r_1, r_2…….. be the distances of the columns from the C.G. of the column group, measured parallel to the direction of the wind.

Force F_1 in column 1 at distance r_1 from C.G. of the column group is given by

$$F_1 = \frac{phr_1}{r_1^2 + r_2^2 + \ldots}$$

$$= \frac{phr_1}{\Sigma r^2}$$

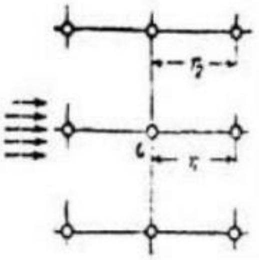


Fig. 17

In the design it is assumed that horizontal shear taken by inner columns is twice that taken by outer columns.

The bracings are designed for B.M. and shear. Same reinforcement is put at top and bottom as the may blow from one side or the other.

Circular tanks are sometimes provided with inclined columns. In such cases the vertical component of the force to each column is found as given above. The horizontal shear in each column is given by deducting the sum of horizontal components of the forces in the columns from the wind force and dividing by number of columns.

The moments in the inclined braces meeting at a column can be found as follows-

The axes of moments in column above and below the brace will be at right angles to the direction of the wind. The axes of moments in the two braces will be at right angles to their axes. By completing the triangle of moments, the moments, the moments in the braces can be found.

Let O be the column and OA and OB be the braces meeting at column O. Oa is drawn perpendicular to OA and OB is drawn perpendicular to OB and ab is drawn at right angles to the direction of wind, ab gives moment in the column Oa and Ob will give the moments in braces OA and OB respectively. For moment to be maximum in OA, the wind should blow at right angles to the other brace OB. In such a case triangle of moments will be right angled triangle and side Oa will be hypotenuse.

Foundation for elevated tanks. The foundation for elevated tank columns may be combined foundation in the form of raft or independent footing may be provided for each column.

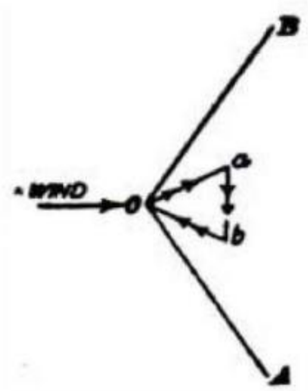


Fig. 18

Ex. 6. Design a circular tank having diameter of 6 m. and height of 3 m. The tank is supported on masonry tower.

Sol. The tank is covered with domed roof. Rise of 1 m. is provided for the dome.

Radius of dome is given by

$$3 \times 3 = 1 \times (2R - 1)$$

$$\therefore \quad R = 5 \text{ m.}$$

Thickness of 10 cm. is provided for dome.

Self-load per square metre

$= 10 \text{ x } 24 = 240$ kg.

Live load per square metre

$= 150$ kg.

Total load = 390 kg/m^2.

Hoop stress at any angle θ is given by

$$\frac{wR}{t} \times \frac{(\cos^2\theta + \cos\theta - 1)}{(1+\cos\theta)}$$

Meridional stress

$$= \frac{(wR(1-\cos\theta)}{t\sin^2}$$

$\sin\phi = \frac{3}{5} = 0.6,$ $\qquad \phi = 36° \ 52'$

$\cos\phi = \frac{4}{5}$

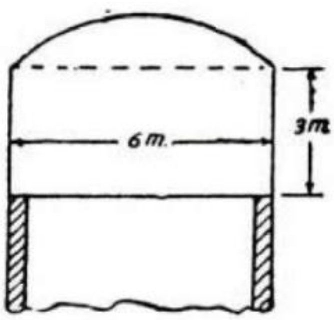


Fig. 9 (a)

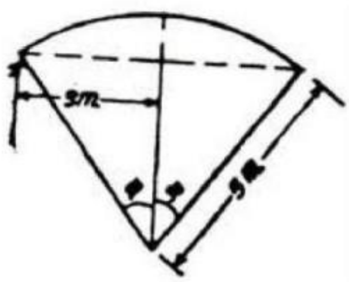


Fig. 9 (b)

$$\text{Meridional stress} = \frac{390 \times 5}{0.1}\ \frac{(1-\cos\theta)}{\sin^2\theta} \text{ x } 10^{-4} \text{ kg/cm}^2$$

$$= 1.95 \text{ x } \frac{(1-\cos\theta)}{\sin^2\theta}$$

$$\text{Hoop stress} = \frac{390 \times 5\,(\cos^2\theta + \cos\theta - 1)}{0.1(1+\cos\ \theta)} x10^{-4}\ kg/cm^2$$

$$= \frac{1.95\,(\cos^2\theta + \cos\theta - 1)}{(1+\cos\ \theta)}\ kg/cm^2.$$

Maximum meridional stress occurs at

$\theta = \phi = 36°52'$.

$$\text{Meridional stress} = 1.95 \times \frac{(1-0.8)}{0.6 \times 0.6}$$

$=1.08$ kg/ cm^2.

Maximum hoop stress occurs at

$\theta = 0°$

$$\text{Hoop stress} \quad = 1.95 \times \frac{(1+1-1)}{1+1}$$

$= 0.975$ kg/cm^2.

Stresses are very small. Provide nominal reinforcement of 8 mm ϕ bars at 20 cm. centres both ways.

To resist the horizontal component of the meridional thrust at the end of done a ring is provided. The ring beam will be subjected to hoop tension.

Design of Ring Beam. Meridional thrust per 1 cm. length

$= 1.08 \times 10 \times 1 = 10.8$ kg

Horizontal component of meridional thrust

$= 10.8 \cos\phi$

$= 10.8 \times 0.8$.

$$\text{Hoop tension} \quad = \frac{pd}{2} = 10.8 \times 0.8 x \frac{600}{2}$$

$= 2592$ kg.

Area of steel required

$$= \frac{2592}{1400} = 1.85\ cm^2.$$

Provide 4 bars of 10 mm ϕ. Area provided

$= 3.14$ cm^2.

6 mm ϕ stirrups at 20 cm. centres are provided.

20 cm x 15 cm. section of ring beam is provided.

Stress in concrete. Equivalent area

$= 20 \times 15 + 12 \times 3.14$

$= 300 + 37.68$

$= 337.68$ cm^2.

$$\text{Tensile stress} \quad = \frac{2592}{337.68} = 7.68\ kg/cm^2.\ \text{Safe.}$$

Design of Cylindrical Walls. As the dome is designed on membrane theory, the tank wall is assumed free at top.

$H = 3$ m., $D = 6$ m. Assume thickness of tank wall = 15 cm.

$$\frac{H^2}{Dt} = \frac{3^2}{6 \times 0.15} = 10$$

From table 17.2.

Maximum hoop tension

$= 0.608\ wHR$

$= 0.608 \times 1000 \times 3 \times 3$

$= 5472$ kg.

Maximum hoop tension occurs at depth of

$0.6\ H = 1.8$ m. from top.

Maximum - ve *B.M.* $= 0.0122\ wH^3$

$= 0.0122 \times 1000 \times 27$

$= 329.4$ kg. m.

Maximum -ve *B.M.* occurs at base.

Maximum +ve *B.M.* $= 0.0029 \times 1000 \times 27$

$= 78.3$ kg. m.

Maximum + ve *B.M.* occurs at depth of

$0.7\ H = 2.1$ m. from top.

Maximum shear at base

$= 0.158\ wH^{2ss}$

$= 0.158 \times 1000 \times 9$

$= 1422$ kg.

Steel required for hoop tension

$$= \frac{5472}{1000}$$

$= 5.472\ cm^2$.

Provide 8 mm ϕ bars at 18 cm. centres on both faces.

Minimum steel required

$$= 0.3 - \frac{0.1 \times (15 - 10)}{(45 - 10)}$$

$= 0.286\%$

Steel required $= \dfrac{0.286 \times 15 \times 100}{100} = 4.29\ cm^2$.

Provide 8 mm ϕ bars at 18 cm. centres throughout the height.

Steel required for - ve *B.M.*

Effective depth = 15 - 4 = 11 cm.

$$A_t = \frac{329.4 \times 100}{0.84 \times 11 \times 1000} = 3.55 \text{ cm}^2$$

Provide 8 mm ϕ bars at 14 cm centres. Every third bar will be stopped at 1 m. from base, the remaining bars will be taken upto top to serve as distribution steel. On the other face 8 mm ϕ bars at 21 cm. centres are provided. The reinforcement will resist +ve B.M.

Design of bottom slab. The bottom slab will be treated as having edges clamped. Assume 22 cm. thickness of slab.

Self load of slab = 0.22 x 1 x 1x 2400 = 528 kg/m^2.
Load of water = 3 x 1000 = 3000 kg/m^2.
Total load = 3000 + 528 = 3528 kg/m^2.

In circular slab of radius '*a*' and uniformly loaded with load of intensity '*q*'.
Circumferential moment

$$= \frac{q}{16}a^2.$$

Radial moment +ve $= \frac{q}{16}a^2$.

- ve $= \frac{2q}{16}a^2$.

Maximum radial negative moment

$$= \frac{2 \times 3528 \times 3^2}{16} = 3969 \text{ kg. m.}$$

Effective depth required

$$= \sqrt{\frac{3969 \times 100}{14.11 \times 100}} \text{ (tension on water side).}$$

= 16.8 cm

Provide overall depth of 22 cm. with effective depth of 18 cm.

$$A_t = \frac{3969 \times 100}{0.84 \times 18 \times 1000} = 26.2 cm^2$$

Provide 16 mm ϕ bars at 7 cm. centres 1.2 m from edge.

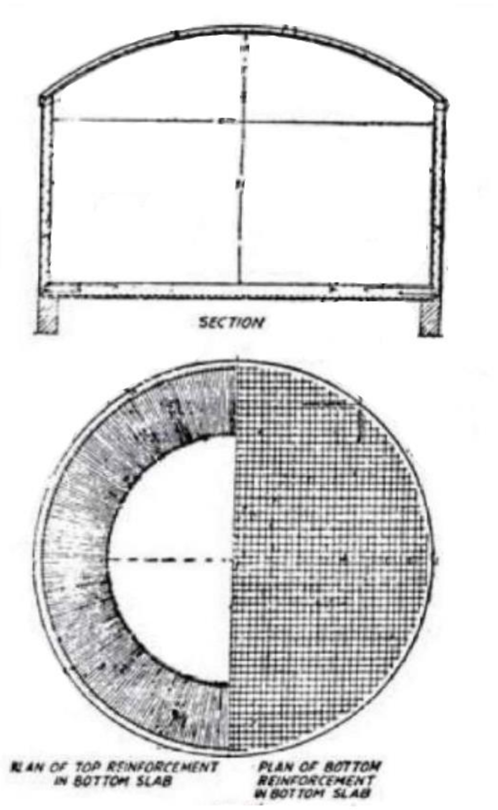


Fig. 19

+ ve radial moment $= \frac{q}{16} a^2.$

$$= \frac{3528 \times 3^2}{16} = 1984.5 \text{ kg. m.}$$

Circumferential moment

$$= 1984.5 \text{ kg. m.}$$

Area of steel $= \frac{1984.5 \times 100}{0.84 \times 18 \times 1000}$ (tension outside) $= 13.0 \text{ cm}^2$

Provide 12 mm ϕ bars at 8 cm. centres both way throughout.

Elevated Rectangular Tanks. Design of elevated tanks open at top is similar to the tanks resting on the ground. If the tank is covered at the top the vertical walls are considered to be supported at the top. In such cases if the ratio of width of tank wall to height of tank is between 0.5 to 2, the coefficients for moments in two directions vertical and horizontal can be obtained from Table 5. The walls are designed for these moments. In case the width of the tank wall is greater than twice the height of tank it will be designed spanning vertically, simply supported at top and fixed at the base. In case the width of tank wall is less than half the height of tank, it is designed spanning horizontally and fixed at the junctions.

The roof slab is designed as two-way slab if the length does not exceed twice the breadth of the tank. In case the length of tank is more than twice the breadth of the tank, the slab is designed oneway spanning along the shorter span.

The design of bottom slab depends on the system of columns and beams provided.

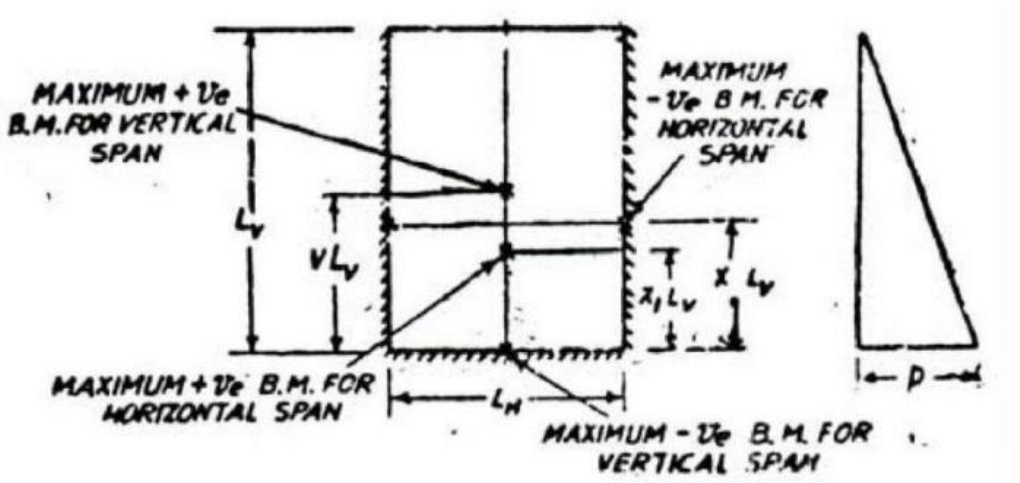


Fig. 20

$B.M. = kpl^2$

where l is the span,

k is coefficient given in Table 5

and p is pressure, $p = w \times Lv$.

For $\frac{L_H}{L_V} > 2$, whole load will act along the vertical span and

maximum -ve $B.M. = \frac{plv^2}{15}$ and + ve $B.M. = \frac{plv^2}{33.5}$

Table 5

$\frac{L_H}{L_V}$	X	X_1	V	Max. -ve B.M. for vertical span	Max. +ve B.M. for vertical span	Max -ve B.M for horizontal span	Max. +ve B.M. for horizontal span
0.5	0.18	0.40	0.25	0.007	0.006	0.050	0.034
1.0	0.33	0.47	0.40	0.026	0.013	0.026	0.012
1.5	0.42	0.48	0.45	.043	0.027	0.013	0.004
2.0	0.45	0.49	0.48	0.055	0.032	0.002	0.001

For $\frac{L_H}{L_V} < 0.5$, whole load will act along the horizontal span and maximum -ve $B.M. = \frac{pL_H^2}{12}$ and maximum + ve $B.M. = \frac{pL^2_H}{16}$

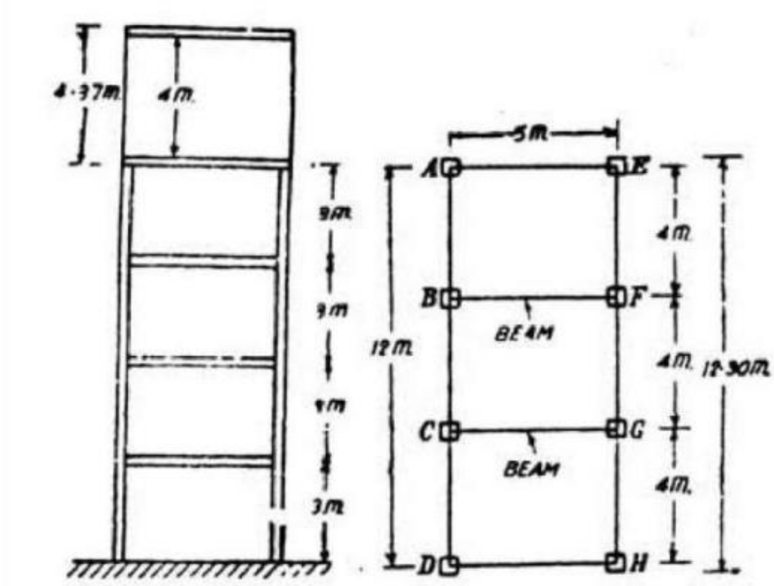


Fig. 21

Ex. 17.7. Design an elevated rectangular tank 12 m. x 5 m. x 4m high. The bottom of tank is 12 m. above ground level. The tank is covered at top.
Bearing capacity of soil is 15,000 kg/m^2.

Sol. The tank will be supported on 8 columns, spaced at 4 m. centres as shown in Fig. 17.21. The columns are braced together at intervals of 3 m.

Concrete mix used is *M* 200.

Design of Roof Slab. Roof slab is designed for a live load of 150 kg/m^2.
Assume thickness of roof slab as 15 cm.

Self load = 0.15 x 1 x 1 x 2400 = 360 kg/m^2.
Live load = 150 kg/m^2.
Total load = 510 kg/m^2.

As the ratio of length to breadth of slab is greater than 2, slab will span in shorter direction.

Maximum *B.M.* $= \frac{wl^2}{8} = \frac{510 \times 5^2}{8}$
= 1593.75 kg. m.
= 159,375 kg. cm.

For mix. $M200$, C = 70, t = 1400, m = 13, Q = 12.10.

Effective depth $= \sqrt{\dfrac{159.375}{12.1 \times 100}} = 11.48$ cm.

Provide overall depth of 15 cm. with effective depth of 13 cm.

$$A_t = \frac{159,375}{1400 \times 0.87 \times 13}$$
$$= 10.07 \text{ cm}^2.$$

Provide 12 mm ϕ bars at 11 cm. centre.

$$\text{Distribution steel} = \frac{0.15 \times 15 \times 100}{100}$$
$$= 2.25 \text{ cm}^2.$$

Provide 6 mm ϕ bars at 12 cm centres.

Lecture 12

REFERENCE:

1. Steel Water Storage Tanks: Design, Construction, Maintenance, and Repair 1st Edition, Kindle Edition by Steve Meier, American Water Works Association.

2. Elementary Structural Design & Drawing (In 3 Vols.) Vol. I: Structural Design & Drawing Kindle Edition by D. Krishnamurthy (Author) Format: Kindle Edition

3. Structural Design & Drawing-Vol. 3 [Print Replica] Kindle Edition by D. Krishnamurthy (Author) Format: Kindle Edition

4. The Dock Manual: Designing/Building/MaintainingJan 4, 1999 by Max Burns

5. Designing Steel Structures for Fire SafetyMay 6, 2009 by Jean Marc Franssen and Venkatesh Kodur

6. Reference Book: Design of Steel Structures by S K Duggal