UNIT 22 WATER TANKS

Structure

22.1 INTRODUCTION

Tank (or reservoir) is a liquid storage structure that can be below or above the ground level. The liquid to be stored may be water, liquid petroleum, petroleum products or similar liquids. Though the tanks can be made of Reinforced Concrete (RC), steel or synthetic materials but in this unit only RC tanks have been covered. The tanks are often classified based on the following features :

(i) Shape : Circular (flexible or rigid base), Rectangular, Intze, Conical or Funnel etc.

(ii) Location with respect to ground : Underground, Resting on ground, Partially underground and Overhead.

(iii) Capacity : Large, Medium and Small.

The shape has a very important role to play because the structural behaviour of different components of the tank depends upon it. Flexure being predominant in rectangular tanks, its section are heavier in comparison to other shapes. But based on the economic considerations due to its simple form, rectangular tanks are sometimes prefered especially for small capacity.

Reservoir below ground level are normally built to store large quantities of water, whereas, the overhead type are built for direct distribution by gravity flow and are usually of smaller capacity.

The capacity requirement of a tank helps in deciding what type of tank will be suitable.

Objectives

After studying this unit, you should be able to design

- circular and rectangular tanks resting on ground, and
- Intze tank by membrane analysis.

22.2 GENERAL DESIGN CONSIDERATIONS

Besides strength, water tightness is one of the main consideration in the design of RC water tanks. It has to be ensured in their design that the concrete does not crack on the water face. Minimum grade of concrete used in water tanks is M 20. Imperviousness of concrete can be ensured by implementing the following recommendations :

(i) Concrete mix containing well graded aggregate with water cement ratio less than 0.5 be used.

(ii) Concrete should be richer in cement and very well compacted. The quantity of concrete. The upper limit on the quantity of cement is to keep the shrinkage within allowable limits.

(iii) Defects such as segregation and honey combing which are the potential source of leakage be avoided.

The crack of concrete can be controlled by adopting the following measures :

(i) The cracking due to shrinkage and temperature variation can be minimised by keeping the concrete moist and filling the tank as soon as possible.

(ii) Avoid the use of thick timber shuttering that prevents the easy escape of the heat of hydration from the concrete mass.

(iii) Cracking is controlled by increasing the requirement of minimum reinforcement as given in Table 22.1.

Table 22.1 : Minimum Reinforcement

Sl. No.	Nature	Percentage	+
		Mild Steel	HYSD
1.	Dummy concrete with no tension	0.15	0.12
2.	Concrete member with Thickness $\leq$ 100 mm Thickness $\geq$ 450 mm 100 mm$\leq$ Thickness (= t mm, say) $\leq$ 450 mm	0.30 0.20 $0.3 - \left(\frac{t-100}{350}\right) \times 0.1$ (Linear interpolation)	0.24 0.16 $0.24 - \left(\frac{t-100}{350}\right) \times 0.08$ (Linear interpolation)

Plain concrete may fracture at 0.0002 strain, but the provision of minimum reinforcement considerably reduces the level of cracking strain. Spacing of reinforcement should not be more than three times the thickness of a member.

(iv) Use of deformed bars or ribbed steel improves the level of cracking strains in concrete by even distribution and slip minimization.

(v) The cracking of concrete is also kept within allowable limits by reducing the allowable stresses in steel as given in Table 22.2.

Table 22.2 : Allowable Stresses in Reinforcement

Sl.No.	Nature of Stress	Allowable Stress (MPa)	
		Plain Mild Steel*	HYSD
1.	Tension in steel placed within 225 mm from water face	100	150
2.	Tension in steel placed beyond 225 mm from water face	120	190
3.	Compression	120	190

*For deformed bars, increase by 20%.

(vi) To keep the concrete free from cracks, the tensile stress in concrete due to direct tension be limited to 0.27 $\sqrt{f_{ck}}$ MPa and due to bending tension to 0.37 $\sqrt{f_{ck}}$ MPa, where f_{ck} is the characteristic compressive strength of 150 mm cube of concrete in MPa. Design criteria for bending and direct tension are given below.

Design Criteria for Bending

Effective depth required, $d = \sqrt{\frac{M}{k\,b}}$

Area of flexural steel required, $A_{st} = \frac{M}{\sigma_{st}\, j\, d}$ (to be provided at flexural tension face)

Design Criteria for Direct Tension

Area of steel required, $A_s = \frac{T}{\sigma_{st}}$ (to be equally distributed in the cross-section)

Thickness of member can be calculated from :

$$\frac{T}{A_c + (m-1) A_s} \leq \sigma_{ct}$$

where, M = Bending Moment (BM)

$k = \frac{1}{2} \sigma_{cbc} \, n \, j$

n = Neutral axis (N. A.) depth coefficient

$= \dfrac{1}{1 + \dfrac{\sigma_{st}}{m \, \sigma_{cbc}}}$

σ_{cbc} = Permissible bending compressive stress for concrete,

σ_{st} = Permissible bending tensile stress for steel,

m = Modular ratio

$= \frac{280}{3 \, \sigma_{cbc}}$,

j = Lever arm coefficient

$= 1 - \frac{n}{3}$,

σ_{ct} = Permissible direct tensile stress for concrete,

T = Direct tension,

A_c = Area of concrete

$= b \, t$,

b = Width of the section, and

t = Thickness of the section.

(vii) The concrete cover to the reinforcement is kept more for controlling the cracking. The minimum recommended cover is :

Clear cover = 20 mm or diameter of bar for steel in direct tension

= 25 mm for steel in bending tension

= 30 mm for alternate drying and wetting condition

SAQ 1

Describe the general design considerations of reinforced water tanks.

22.3 JOINTS IN WATER TANKS

The various types of joints that are provided in water tanks can be categorized under three heads :

(i) Movement Joints

(ii) Construction Joints

(iii) Temporary Open Joints

22.3.1 Movement Joints

These are provided for accommodating relative movement of the two sides. All movement joints are essentially flexible joints and require the incorporation of special materials in order to maintain water tightness. The movement joints are of three types :

(i) Contraction Joint

(ii) Expansion Joint

(iii) Sliding Joint

(i) *Contraction Joints*

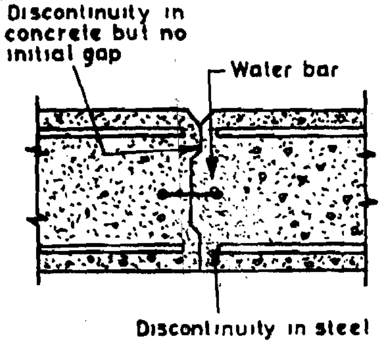
A contraction joint is a typical movement joint that accommodates the contraction of concrete due to shrinkage and thermal effects. The joint may be either a complete contraction joint (Figure 22.1(a)) in which there is discontinuity of both concrete and steel, or it may be a partial contraction joint in which there is a discontinuity of concrete but the steel runs through the joint (Figure 22.1(b)). In both cases, no initial gap is kept at the joint but only discontinuity is given during construction. In the complete contraction joint, a water bar is provided whereas in partial contraction joint, the mouth of the joint is filled with joint sealing compound and then strip painted. A water bar is pre-formed strip of impermeable material such as metal, PVC or rubber. Joint sealing compounds are impermeable ductile materials that provide a water-tight seal by adhesion to the concrete throughout the range of joint movement. The commonly used materials are based on asphalt, bitumen or coaltar with or without fillers such as limestone or slate dust, asbestos fibre, chopped hemp, rubber or other suitable material. These are usually applied after the construction or just before the reservoir is put into service by pouring in the hot or cold state, by trowelling or gunning or as pre-formed strips ironed into position.

(ii) *Expansion Joint*

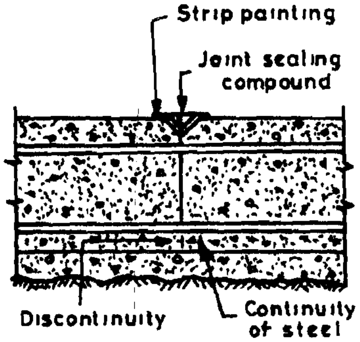
Expansion joint is provided to accommodate either expansion or contraction of the structure. It is a movement joint with complete discontinuity in both concrete and steel (Figure 22.1(c)). An initial gap is usually kept between the adjoining parts of a structure which by closing or opening accommodates the expansion or contraction of the structure. The initial gap is filled with a joint filler. Joint fillers are normally compressible sheet or strip of materials used as spacers. Water bar is provided for water tightness.

(iii) *Sliding Joint*

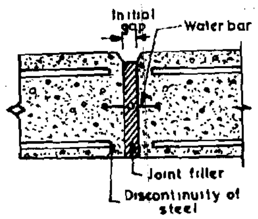
It is a movement joint with complete discontinuity in both concrete and steel and is intended to facilitate the relative movement. A typical application of such a joint is between the wall and floor of cylindrical tanks with flexible base (Figure 22.1(d)).



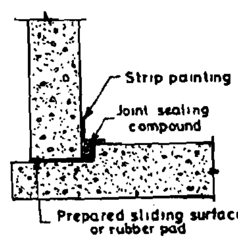
(a) Complete Contraction Joint



(b) Partial Contraction Joint



(c) Typical Expansion Joint



(d) Typical Sliding Joint

22.3.2 Construction Joint

A construction joint is a joint introduced for convenience in the construction. Special measures are taken to achieve continuity at the joint to avoid any relative movement. It is, therefore, a rigid joint in contrast to a movement joint which is a flexible joint. A typical construction joint between successive lifts in the wall of a reservoir is shown in Figure 22.2. The position and arrangement of all construction joints should be predetermined by the engineer. Consideration should be given to limiting the number of such joints.

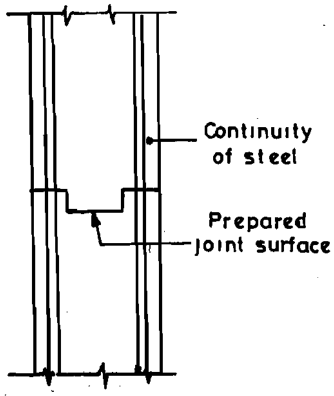


Figure 22.2 : Construction Joint

22.3.3 Temporary Open Joint

In this type of joint, a gap temporarily left between the concrete of adjoining parts of a structure which after a suitable interval and before the structure is put into use, is filled with mortar or concrete either completely (Figure 22.3(a)) or with the inclusion of suitable joining material (Figure 22.3(b) and (c)). In the former case the gap should be sufficient to allow the sides to be prepared before filling.

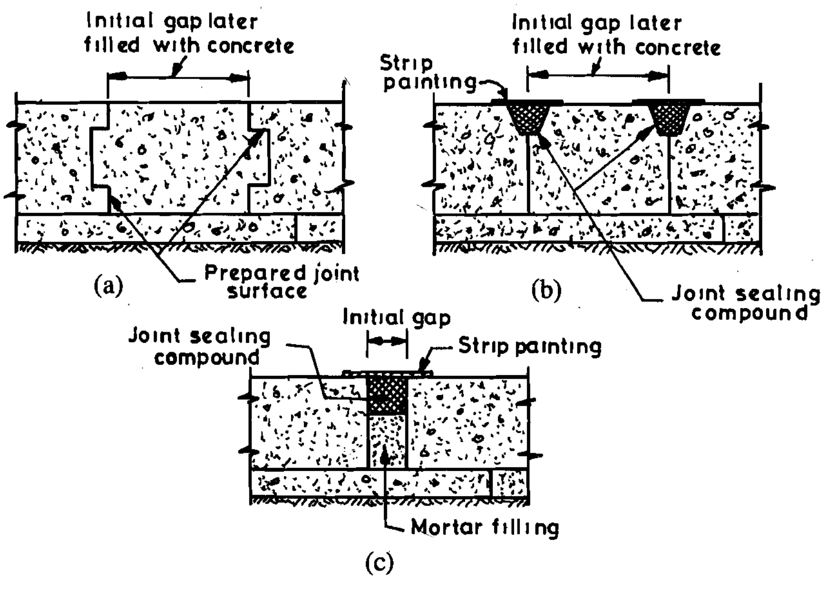


Figure 22.3 : Typical Temporary Open Joints

SAQ 2

Describe the various type joints used in water tanks.

22.4 DESIGN OF TANKS

The behaviour and design procedure of various tanks has been discussed under the following heads :

(i) Tanks Resting on Ground

(ii) Overhead Tanks

(iii) Underground Tanks

22.4.1 Tanks Resting on Ground

The water tanks resting on ground may be of the following types :

(i) Circular Tank with Flexible Base

(ii) Circular Tank with Rigid Base

(iii) Rectangular Tank

If the floor slab is resting continuously on the ground, a minimum thickness of 150 mm may be provided with a nominal reinforcement of 0.24% HYSD steel bars (0.3% for MS bars) in each direction. The slab should rest on a 75 mm thick layer of lean concrete (M 10 mix). The layer of lean concrete should be first cured and then it should be covered with a layer of tarfelt to enable the floor slab to act independent of the bottom layer of concrete.

The design of tank walls of various types has been discussed in the following sections :

(i) **Circular Tank with Flexible Base**

Due to water pressure, the wall of a circular tank with flexible base between wall and base slab expand circumferentially which increases linearly from zero at top to a maximum at the base as shown in Figure 22.4 by dotted line AB'. The circumferential expansion causes only hoop tension in the wall which will also increase linearly from top to the base of the wall. If D is the internal diameter of the tank, hoop tension at a depth h will be $\gamma\, h \frac{D}{2}$, here γ is the unit weight of water. This will be zero at the top of the wall where h is zero and maximum at the base of the wall. Example 22.1 explains the design procedure of this tank.

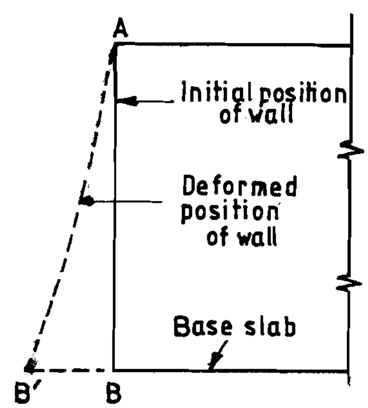


Figure 22.4 : Circular Tank with Flexible Base

Example 22.1

Desgin an open circular water tank resting on firm ground with flexible base for 350 kl capacity.

Solution

(1) *Proportioning of the Tank*

Assuming the height of wall (inside) as 4.0 m.

Keeping 200 mm free board, effective depth of water, $H = 4.0 - 0.2 = 3.8$ m

Let D be the inside diameter of the tank in meter.

Capacity of the tank $= \frac{\pi}{4}\, D^2 H = 350 \text{ m}^3$

or, $\quad \frac{\pi}{4} D^2 \times 3.8 = 350$

or, $\quad D = 10.83$ m

Providing 11.0 m diameter ($r = 5.5$ m).

(2) *Allowable Stresses*

M 20 grade concrete and Fe 415 grade steel (HYSD) will be used.

Permissible direct tensile stress of concrete, $\sigma_{ct} = 1.2$ MPa

Permissible tensile stress of steel,

$$\sigma_{st} = 150 \text{ MPa upto 225 mm from water face}$$

$$= 190 \text{ MPa beyond 225 mm from water face}$$

(3) *Design of Wall*

Height of wall, $h = 4.0$ m

Hoop tension at the base of the wall, $T_1 = \gamma\, r\, h = 10 \times 5.5 \times 4.0 = 220$ kN

Hoop tension at 3.0 m from top, $\quad T_2 = 10 \times 5.5 \times 3.0 = 165$ kN

Total hoop tension in the bottom 1 m depth, $T = \frac{1}{2}(T_1 + T_2) = 192.5$ kN

Area of steel required in the bottom 1 m depth

$$A_s = \frac{T}{\sigma_{st}} = \frac{192.5 \times 10^3}{150} = 1284 \text{ mm}^2$$

Provide 7 ϕ 16, A_s (provided) = $7 \times 201 = 1407$ mm^2

Tensile stress in concrete = $\frac{T}{A_c + (m-1)A_s} \leq \sigma_{ct}$

or, $\quad \frac{192.5 \times 10^3}{A_c + (13-1) \times 1407} \leq 1.2$ or, $A_c = 143533$ mm^2

Thickness required, $t = \frac{A_c}{1000} = 143.5$ mm

Provide the thickness of wall as 160 mm at the base and tapered to 100 mm at the top.

Average thickness between 3.0 and 4.0 m depth = $100 + \frac{160 - 100}{4} \times 3.5$

$$= 152.5 \text{ mm} > 143.5 \text{ mm}$$

The hoop reinforcement in vertical wall is given in Table 22.3.

Table 22.3 : Hoop Reinforcement in Vertical Wall

Distance from Top (m)	Hoop Tension T (kN)	Reinforcement	
		Required (mm^2)	Number of ϕ 12 Bars
0.0 to 1.0	27.5	183	4
1.0 to 2.0	82.5	550	5
2.0 to 3.0	137.5	917	9
3.0 to 4.0	192.5	1284	12

Average thickness of wall = 130 mm

Minimum percentage of steel for 130 mm thickness

$$= 0.24 - \frac{(130 - 100)}{350} \times 0.08$$

$$= 0.233$$

Minimum vertical steel required = $\frac{0.233}{\ldots} \times 1000 \times 130 = 303$ mm^2

Provide $\phi 10$ @ 250 mm c/c vertical steel.

As the thickness of wall is less than 225 mm, therefore, reinforcement has been provided in the middle of the thickness of wall.

(4) *Design of Base Slab*

As the slab is resting on the ground, provide a nominal thickness of 150mm.

Minimum steel, $A_s = \frac{0.24}{100} \times 1000 \times 150 = 360 \text{ mm}^2/\text{ m}$ in each direction

Provide half steel on each face.

Provide ϕ 8 @ 250 mm c/c in both directions and at top and bottom.

The reinforcement detail is shown in Figure 22.5.

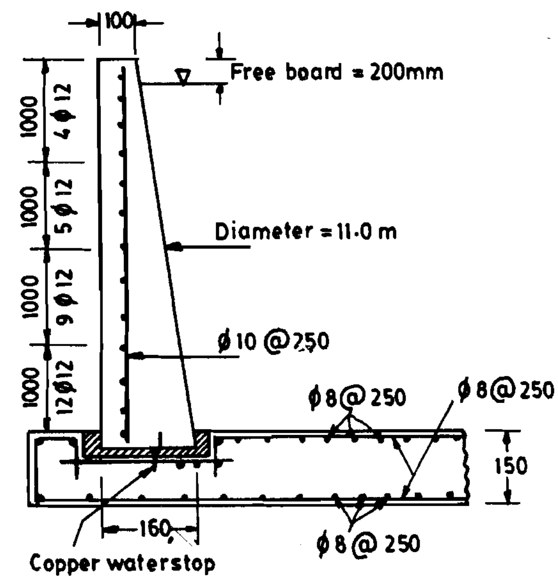


Figure 22.5 : Reinforcement in Tank Wall and Base Slab (All dimensions are in mm)

(ii) Circular Tank with Rigid Base

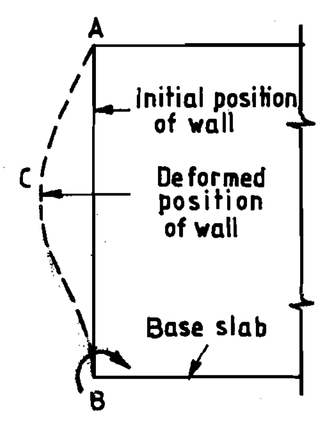
When the joint between the wall and face is rigid, there will not be any circumferential movement of the wall at its base and the wall will take the shape ACB (Figure 22.6). The upper portion of the wall will have hoop tension while the lower part will bend like a cantilever fixed at B. In this case hoop tension will not be maximum at the base as in circular tank with flexible base. There are various methods for the determination of cantilever BM and hoop tension along the height of the wall. These methods are :

(a) Reissner's Method

(b) Carpenter's Simplified Method

(c) BIS Code Method

(d) Approximate Method



(a) *Reissner's Method*

According to this method the behaviour of circular tank with rigid base depends upon a parameter k defined by

$$k = \frac{48\,H^4}{D^2\,t^2}$$

where, H = Height of wall,

D = Inside diameter of tank, and

t = Thickness of wall of the tank.

Reissner developed tables from which cantilever BM at the base of the wall (Table 22.4), maximum hoop tension and its location (Table 22.5) are given in terms of the parameter k. The values are given for rectangular as well as triangular wall. Thus for a tepered wall of trapezoidal section a combination of the two may be considered. The maximum positive cantilever BM may be taken approximately 30% of the restraint BM i.e., cantilever BM at the base.

Table 22.4 : Reissner's Value of Restraint Moment

K	**Rectangular Wall Section**		**Triangular Wall Section**	
	Max. Tension	**Height from Base**	**Max. Tension**	**Height from Base**
0	0	—	—	—
10	$0.13\,P\left(\frac{D}{2}\right)$	$10\,H$	$0.09\,P\left(\frac{D}{2}\right)$	$0.65\,H$
100	$0.27\,P\left(\frac{D}{2}\right)$	$1.0\,H$	$0.31\,P\left(\frac{D}{2}\right)$	$0.58\,H$
1000	$0.47\,P\left(\frac{D}{2}\right)$	$0.47\,H$	$0.52\,P\left(\frac{D}{2}\right)$	$0.44\,H$
10000	$0.67\,P\left(\frac{D}{2}\right)$	$0.31\,H$	$0.70\,P\left(\frac{D}{2}\right)$	$0.30\,H$
∞	$1.0\,P\left(\frac{D}{2}\right)$	0	$1.0\,P\left(\frac{D}{2}\right)$	0

Table 22.5 : Reissner's Value of Hoop Tension

K	**Rectangular Wall Section**	Triangular Wall Section
0	$0.167\,pH^2$	$0.167\,pH^2$
10	$0.110\,pH^2$	$0.140\,pH^2$
100	$0.0582\,pH^2$	$0.0707\,pH^2$
1000	$0.024\,pH^2$	$0.026\,pH^2$
10000	$0.0085\,pH^2$	$0.009\,pH^2$
∞	0	0

(b) *Carpenter's Simplified Method*

Carpenter simplified the Reissner's method and gave the following expressions for the calculation of maximum cantilever BM, maximum hoop tension and its position :

Maximum cantilever BM, $M = F\,\gamma\,H^3$

Position of maximum Hoop Tension $= k\,H$ above base

Maximum Hoop Tension $T = \gamma\,(H - K\,H)\,\frac{D}{}$

The values of the coefficients F and K depend upon H/D and H/t ratios, and may be taken from Table 22.6.

Table 22.6 : Carpenter's Value of Coefficients F and K

Factor		F				K			
$\frac{H}{t} \rightarrow$		10	20	30	40	10	20	30	40
Values of H/D	0.2	0.046	0.028	0.022	0.015	—	0.50	0.45	0.40
	0.3	0.032	0.019	0.014	0.010	0.55	0.43	0.38	0.33
	0.4	0.024	0.014	0.010	0.007	0.50	0.39	0.35	0.30
	0.5	0.020	0.012	0.009	0.006	0.45	0.37	0.32	0.27
	1.0	0.012	0.006	0.005	0.003	0.37	0.30	0.24	0.21
	2.0	0.006	0.003	0.002	0.002	0.28	0.22	0.19	0.16
	4.0	0.004	0.002	0.002	0.001	0.27	0.20	0.17	0.14

(c) *BIS Code Method*

Bureau of Indian Standard Code IS 3370 (Part IV) -1967 gives tables for BM and hoop tension in circular tanks for various condition of joints and various types of loading. However, in this section only the case of circular tank with rigid base subjected to triangular water pressure will be discussed. Table 22.7 gives the coefficients for hoop tension at various height in the wall for various values of $\frac{H^2}{Dt}$ ratio. The hoop tension is calculated from the expression :

$$\text{Hoop tension, } T = (\text{coeff.})\ \gamma H \frac{D}{2}$$

Table 22.7 : Coefficient for Tension in Cylindrical Wall Fixed to the Base

$\frac{H^2}{Dt}$	Coefficient at Point									
	0.0 H	0.1 H	0.2 H	0.3 H	0.4 H	0.5 H	0.6 H	0.7 H	0.8 H	0.9 H
0.4	+ 0.149	+ 0.134	+ 0.120	+ 0.101	+ 0.082	+ 0.066	+ 0.049	+ 0.029	+ 0.014	+ 0.004
0.8	+ 0.263	+ 0.239	+ 0.125	+ 0.109	+ 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	+ 0.265	+ 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	+ 0.330	+ 0.262	+ 0.157	+0.052
4.0	+ 0.067	+ 0.164	+ 0.256	+ 0.339	+ 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	+ 0.369	+ 0.398	+ 0.259	+ 0.092
6.0	+ 0.018	+ 0.119	+ 0.234	+ 0.344	+ 0.441	+ 0.504	+ 0.514	+ 0.447	+ 0.301	+ 0.112
8.0	− 0.001	+ 0.104	+ 0.218	+ 0.355	+ 0.445	+ 0.534	+ 0.575	+ 0.530	+ 0.381	+ 0.151
10.0	− 0.003	+ 0.098	+ 0.208	+ 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	+ 0.99	+ 0.0199	+ 0.304	+ 0.412	+ 0.531	+ 0.641	+ 0.687	+ 0.582	+ 0.265

Note : Positive sign indicates tension.

Table 22.8 : Coefficient for Moment in Cylindrical Wall Fixed at Base

$\frac{H^2}{Dt}$	Coefficient at Point									
	0.1 H	0.2 H	0.3 H	0.4 H	0.5 H	0.6 H	0.7 H	0.8 H	0.9 H	0.10 H
0.4	+ 0.0005	+ 0.0014	+ 0.0021	+ 0.0007	+ 0.0042	− 0.0150	− 0.0302	− 0.0529	− 0.0816	− 0.1205
0.8	+ 0.0011	+ 0.0037	+ 0.0063	+ 0.0080	+ 0.0070	+ 0.0023	− 0.0068	− 0.0024	− 0.0465	− 0.0795
1.2	+ 0.0012	+ 0.0042	+ 0.0077	+ 0.0103	+ 0.0112	+ 0.0090	+ 0.0022	− 0.0108	− 0.0311	− 0.0602
1.6	+ 0.0011	+ 0.0041	+ 0.0075	+ 0.0107	+ 0.0121	+ 0.0111	+ 0.0058	− 0.0051	− 0.0232	− 0.0505
2.0	+ 0.0010	+ 0.0035	+ 0.0068	+ 0.0099	+ 0.0120	+ 0.0115	+ 0.0075	− 0.0021	− 0.0185	− 0.0436
3.0	+ 0.0006	+ 0.0024	+ 0.0047	+ 0.0071	+ 0.0090	+ 0.0097	+ 0.0077	+ 0.0012	− 0.0119	− 0.0333
4.0	+ 0.0003	+ 0.0015	+ 0.0028	+ 0.0047	+ 0.0066	+ 0.0077	+ 0.0069	+ 0.0023	− 0.0080	− 0.0268
5.0	+ 0.0002	+ 0.0008	+ 0.0016	+ 0.0029	+ 0.0046	+ 0.0059	+ 0.0059	+ 0.0028	− 0.0058	− 0.0222
6.0	+ 0.0001	+ 0.0003	+ 0.0008	+ 0.0019	+ 0.0032	+ 0.0046	+ 0.0051	+ 0.0029	− 0.0041	− 0.0187
8.0	0.0000	+ 0.0001	+ 0.0002	+ 0.0008	+ 0.0016	+ 0.0028	+ 0.0038	+ 0.0029	− 0.0022	− 0.0146
10.0	0.0000	+ 0.0000	+ 0.0001	+ 0.0004	+ 0.0007	+ 0.0019	+ 0.0029	+ 0.0028	− 0.0012	− 0.0122
12.0	0.0000	− 0.0001	+ 0.0001	+ 0.0002	+ 0.0003	+ 0.0013	+ 0.0023	+ 0.0026	− 0.0005	− 0.0104
14.0	0.0000	0.0000	0.0000	0.0000	+ 0.0001	+ 0.0008	+ 0.0019	+ 0.0022	− 0.0001	− 0.0090
16.0	0.0000	0.0000	− 0.0001	− 0.0002	− 0.0001	+ 0.0004	+ 0.0013	+ 0.0019	+ 0.0001	− 0.0079

(**Note :** Positive sign indicates tension on the outside.)

Table 22.8 gives the coefficient for vertical BM at various height in the wall for various values of $\frac{H^2}{D\,t}$ ratio. The BM is given by :

$$BM = (\text{Coeff.})\ \gamma\ H^3$$

Table 22.9 gives the coefficients for shear at the base of the wall. The shear is determined from the expression :

$$\text{Shear at the base of wall} = (\text{Coeff.})\ \gamma\ H^2$$

Table 22.9 : Coefficients for Shear at the Base

$\frac{H^2}{Dt}$	Coefficient	$\frac{H^2}{Dt}$	Coefficient
0.4	+ 0.436	5.0	+ 0.213
0.8	+ 0.374	6.0	+ 0.197
1.2	+ 0.339	8.0	+ 0.174
1.6	+ 0.317	10.0	+ 0.158
2.0	+ 0.299	12.0	+ 0.145
3.0	+ 0.262	14.0	+ 0.135
4.0	+ 0.236	16.0	+ 0.127

(d) *Approximate Method*

In Figure 22.7, triangle ABC shows hydrostatic water pressure on wall AB. In the approximate method, it is assumed that cantilever action will be in a height *h* above base i.e., height BD. The assumed distribution of load causing cantilever bending and hoop action is shown in Figure 22.7, thus showing that the hoop tension will be maximum at the height h above base i.e., at point D. The value of height h is calculated approximately as follows :

$$h = \frac{H}{3} \text{ or 1 m (whichever is more)for } 6 \leq \frac{H^2}{D\,t} \leq 12$$

$$= \frac{H}{4} \text{ or 1 m (whichever is more)for } 12 < \frac{H^2}{D\,t} \leq 30$$

The cantilever BM and hoop tension can now be easily calculated at any height from the load distribution diagram. Their maximum values will be :

Maximum cantilever BM, $\quad M = \gamma\, H\, \frac{h^2}{6}$

Maximum hoop tension at D, $\quad T = \gamma\, (H - h)\, \frac{D}{2}$

The reinforcement for cantilever BM will have to be provided on water face, whereas, hoop reinforcement will be in the form of rings to be provided either in the middle of the thickness of wall or on both faces. A tank has been desinged based on this method in Example 22.2.

Example 22.2

Redesign the tank of Example 22.1, assuming that the base of the wall is monolithic with the base slab.

Solution

(1) *Dimensions of the Tank*

Diameter, D =11.0 m (r =5.5 m)

Height, H = 4.0 m

Providing thickness of wall, t =150 mm

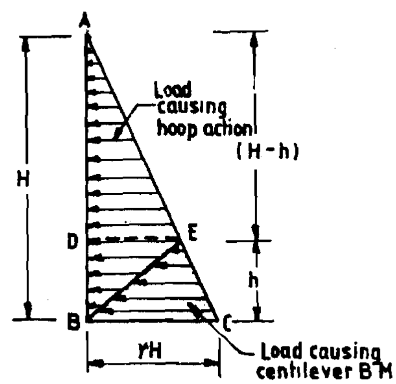


Figure 22.7 : Assumed Load Distribution in Circular Tank with Rigid Base

(2) *Allowable Stresses and Design Constants*

Grade of concrete =M 20

Grade of steel = Fe 415 (HYSD)

Permissible direct tensile stress of concrete, σ_{ct} = 1.2 MPa

Permissible bending compressive stress of concrete, σ_{cbc} = 7.0 MPa

Modular ratio, $m = \frac{280}{3\,\sigma_{cbc}} = 13$

Permissible tensile stress of steel,

σ_{st} = 150 MPa upto 225 mm from water face

= 190 MPa beyond 225 mm from water face

N. A. depth coefficient, $n = \dfrac{1}{1+\dfrac{\sigma_{st}}{m\,\sigma_{cbc}}} = \dfrac{1}{1+\dfrac{150}{13\times 7}} = 0.378$

Lever arm coefficient, $j = 1 - n/3 = 0.874$

$$k = \frac{1}{2}\sigma_{cbc}\, n\, j = 1.155$$

(3) *Design of Wall for Cantilever Action*

Employing approximate method of analysis.

$$\frac{H^2}{D\,t} = \frac{4.0^2}{11.0 \times 0.15} = 9.7$$

Cantilever action will be upto h, where, $h = H/3$ or 1 m whichever is greater.

$$\therefore\ h = \frac{4}{3} = 1.333 \text{ m}$$

Maximum cantilever BM,

$$M = \frac{1}{6}\gamma H h^2 = \frac{1}{6} \times 10 \times 4.0 \times 1.333^2 = 11.85 \text{ kN m/m}$$

Effective thickness of wall required = $\sqrt{\dfrac{M}{Kb}} = \sqrt{\dfrac{11.85 \times 10^6}{1.155 \times 1000}} \approx 102$ mm

Effective thickness allowable, $d = 150 - 35 = 115$ mm > 102 mm (O. K.)

Area of vertical steel required,

$$A_s = \frac{m}{\sigma_{st}\, j\, d} = \frac{11.85 \times 10^6}{150 \times 0.874 \times 115} = 786 \text{ mm}^2\text{/m}$$

Provide ϕ12 @ 140 mm c/c ($A_s = 807$ mm^2) upto 1.4 m from base and above this height curtail half of the bars. So, steel in the top 2.6 m depth will be ϕ 12 @ 280 mm c/c ($A_s = 404$ mm^2). These vertical bars are to be provided at inner face.

Minimum percentage of steel for 150 mm thickness

$$= 0.24 - \frac{150 - 100}{350} \times 0.08 = 0.23$$

Minimum steel = $\dfrac{0.23}{100} \times 1000 \times 150 = 345 \text{ mm}^2 < 404 \text{ mm}^2$ (O. K.)

(4) *Design of Wall for Hoop Tension*

Maximum hoop tension,

$T_1 = \gamma\, r\,(H - h) = 10 \times 5.5 \times (4.0 - 1.333) = 146.7$ kN at 1.333 m from base

Hoop Tension at 2.0 m for top, $T_2 = 10 \times 5.5 \times 2.0 = 110.0$ kN

Total hoop tension between 2.0 m from top to 2.667 m from top,

$$T = \frac{1}{2}(T_1 + T_2)$$

$$= 128.4 \text{ kN}$$

Area of hoop steel, $A_s = \dfrac{T}{\sigma_{st}} = \dfrac{128.4 \times 10^3}{150} = 856 \text{ mm}^2$

(provide 12 ϕ 10, $A_s = 942$ mm^2)

The hoop steel in vertical wall is given in Table 22.10.

Table 22.10 : Hoop Steel in Vertical Wall

Distance from top (m)	Hoop Tension T (kN)	Reinforcement	
		Required (mm2)	Number of ϕ 10 bars
0.0 to 1.0	27.5	183	3 on each face (min)
1.0 to 2.0	82.5	550	4 on each face (min)
2.0 to 2.7	128.4	856	6 on each face (min)
2.7 to 4.0	—	404 (min)	3 on each face (min)

Tensile stress in concrete

$$= \frac{T}{A_c + (m-1)\,A_s} = \frac{128.4 \times 10^3}{667 \times 150 + (13-1) \times 942} = 1.15 \text{ MPa} < 1.2 \text{ MPa (O.K.)}$$

Provide 150 × 150 mm haunches at the junction of wall with the base slab and provide ϕ 8 @ 220 mm c/c in it as shown in Figure 22.8.

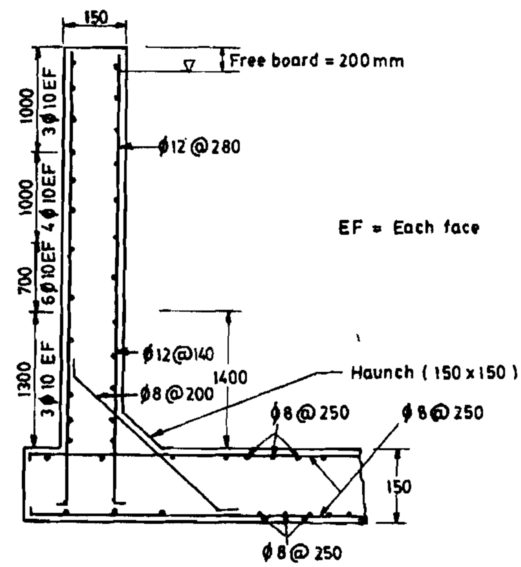


Figure 22.8 : Vertical Section of Tank Showing Reinforcement
(All dimensions in mm)

(5) *Design of Base Slab*

Same as in Example 22.1.

Reinforcement has been shown in Figure 22.8.

(iii) **Rectangular Tank**

For small capacity sometimes a rectangular tank is adopted to avoid exessive expenditure on curved shuttering required for circular water tanks. These tanks, however, are uneconomical for large capacity. The walls of a rectangular water tank are subjected to vertical as well as horizontal BM and pull on some portion of walls. The top edge of the walls which supports a relatively light roof slab can be treated as hinged or free, if the tank is open. The bottom edge of the walls which is normally built integrally with the base slab is treated as fixed. There are situations where master pads, etc. are provided between the wall and the bottom slab, then the joint is treated as hinged.

The analysis of moments in walls of the tank is more difficult as the water pressure applies a triangular load on them. The magnitude of moments will depend upon the relative proportions of length, width and height of the tank and the support conditions of the top and bottom edges of the walls.

The analysis of moments in the walls of a tank is made by elastic theory. The resulting differential equation is not easy to solve and therefore accurate solutions covering all cases are not available. IS 3370 (Part IV) gives tables from which moments and shears in walls for certain edge conditions can be calculated either directly or with suitable modifications. Alternatively, an approximate method can be employed for the design of open rectangular tanks. The method is discussed below.

Approximate Method

Consider an open rectangular tank of dimension L (Length) × B (Breadth) × H (Height). For designing by approximate method, the tank can be divided into two categories :

(a) $L/B \leq 2$

(b) $L/B > 2$

(a) $\boldsymbol{L/B \leq 2}$ **:** In Figure 22.9(a), triangle MPQ shows hydrostatic water pressure on wall MP. It is assumed that the cantilever action will be in a height h above base. The assumed distribution of load causing vertical cantilever bending and horizontal bending is shown in Figure 22.9(a), which shows that the horizontal bending will be maximum at the height h above base. The value of height h is taken as $H/4$ or 1 m whichever is greater. The final horizontal BM is calculated by the analysis of the continuous frame shown in Figure 22.9(b). The moment distribution method can be employed for the purpose. The cantilever BM at the base can be easily calculated as below :

$$\text{Vertical cantilever BM at the base} = \gamma\, H\, \frac{h^2}{6}$$

In addition to the BM, the walls are also subjected to direct tension whose values at height h above base are as follows :

$$\text{Direct tension in long walls} = p\, \frac{B}{2}$$

$$\text{Direct tension in short walls} = p\, \frac{L}{2}$$

where, p = hydrostatic pressure at height h above base

$= \gamma\,(H - h)$

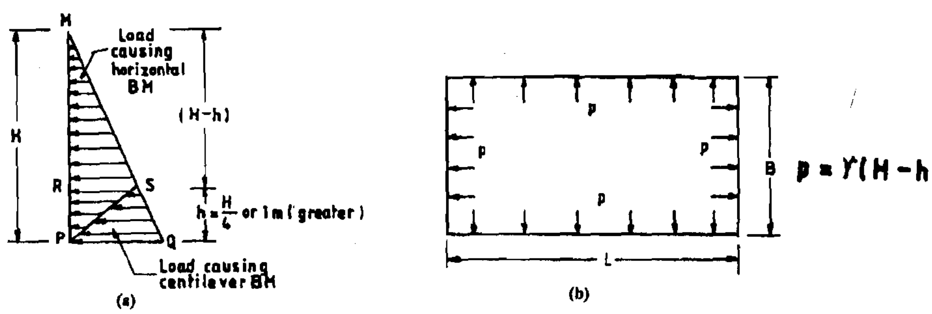


Figure 22.9 : Analysis of Rectangular Tank by Approximate Method ($L/B \leq 2$)

(b) $\boldsymbol{L/B > 2}$ **:** The long walls are assumed to bend vertically as cantilever under the action of triangular hydrostatic pressure. In short walls, cantilever action is assumed in a height h above base. The distribution of load causing cantilever action and horizontal bending is same as for $L/B \leq 2$. The bending moments can thus be calculated easily as given below :

$$\text{Maximum cantilever BM in long wall} = \frac{\gamma\, H^3}{6}$$

$$\text{Maximum cantilever BM in short wall} = \frac{\gamma\, H\, h^2}{6}$$

Maximum horizontal BM at a height h above base in short wall

$$\approx \frac{p\, B^2}{2} \quad \text{at ends as well as mid}$$

In addition to the bending moments, the long and short walls are subjected to direct tension. Since the short walls are assumed to be supported on long walls at its ends, there will be pull in the long walls which will be maximum at height h above base and can be calculated as below :

$$\text{Maximum tension in long wall} = \frac{p\ B}{2} \text{ where, } p = \gamma\ (H - h)$$

As the long walls behave as cantilever, therefore, no pull is transmitted to short wall because of water pressure on long walls. However, the water pressure on the end 1 m width of long wall is assumed to cause direct tension in short wall because cantilever action will not be there close to the ends. Thus,

$$\text{Maximum direct tension in short wall at height } h \text{ above base} = \frac{p\ L'}{2}$$

where, $L' = 2$ m

Design of section subjected to the combined effects of bending and pull is described in the following :

Let M' be the horizontal BM and T be the pull in it. The pull in the wall is transmitted through steel bars thus introducing BM in wall which will thereby reduce the BM. The reduced BM will be

$$M = M' - Tx,$$

where, x = distance of steel from central axis = $d - t/2$,

d = effective depth of section,

t = thinckess of wall.

Area of steel is calculated separately for net BM and for the pull by the following expressions :

$$\text{Area of steel for net BM, } A_{s1} = \frac{M' - Tx}{\sigma_{st}\ j\ d}$$

$$\text{Area of steel for net pull, } A_{s2} = \frac{T}{\sigma_{st}}$$

$$\text{Total steel, } \quad A_s = A_{s1} + A_{s2}$$

The variables used have already been defined.

A design example of a rectangular water tank resting on the ground is presented here which clearly explains the procedure of design by approximate method.

Example 22.3

Design an open rectangular tank 6.6 × 4.2 × 3.3 m deep. The tank rests on firm ground. Employ approximate method of design.

Solution

(1) *Allowable Stresses and Design Constants*

Concrete grade = M 20

Steel grade = Fe 415 (HYSD)

Permissible direct tensile stress of concrete, $\sigma_{ct} = 1.2$ MPa

Permissible bending compressive stress of concrete, $\sigma_{cbc} = 7.0$ MPa

$$\text{Modular ratio, } m = \frac{280}{3\ \sigma_{cbc}} = 13$$

Permissible tensile stress of steel,

$\sigma_{st} = 150$ MPa steel upto 225 mm from water face

$= 190$ MPa steel beyond 225 mm from water face

N. A. depth coefficient, $n = \dfrac{1}{1 + \dfrac{\sigma_{st}}{m\,\sigma_{cbc}}} = \dfrac{1}{\dfrac{150}{13 \times 7}} = 0.378$ for $\sigma_{st} = 150$ MPa

$$= \frac{1}{1 + \dfrac{190}{13 \times 7}} = 0.324 \text{ for } \sigma_{st} = 190 \text{ MPa}$$

Lever arm coefficient, $j = 1 - n/3 = 0.874$ for $\sigma_{st} = 150$ Mpa

$$= 0.892 \text{ for } \sigma_{st} = 190 \text{ MPa}$$

$$k = \frac{1}{2}\sigma_{cbc}\, n\, j = \frac{1}{2} \times 7 \times 0.378 \times 0.874$$

$$= 1.155 \text{ for } \sigma_{st} = 150 \text{ MPa}$$

(2) *BM for Horizontal Bending,* $L = 6.6$ m, $B = 4.2$ m, $H = 3.3$ m

$$\frac{L}{B} = \frac{6.6}{4.2} = 1.57 < 2$$

There will be cantilever action upto a height h above base

$$h = \frac{H}{4} \text{ or } 1 \text{ m whichever is greater}$$

$$= 1.0 \text{ m}$$

In the top 2.3 m height both walls will bend horizontally.

BM in the walls will be determined by moment distribution method.

Water pressure at 2.3 m for top, $p = 10 \times 2.3 = 23$ kN/m^2

Fixed end moment for long wall $= \dfrac{p\,L^2}{12} = \dfrac{23 \times 6.6^2}{12} = 83.5$ kN/m^2

Fixed end moment for short wall $= \dfrac{p\,B^2}{12} = \dfrac{2.3 \times 4.2^2}{12} = 33.8$ kN/m^2

Distribution factor for long wall $= \dfrac{1/6.6}{1/6.6 + 1/4.2} = 0.389$

Distribution factor for short wall $= \dfrac{1/4.2}{1/6.6 + 1/4.2} = 0.611$

Final BM at corners $= 83.5 - 0.389 \times (83.5 - 33.8)$

$= 64.17$ kN m/m (tension at water face)

BM at mid of long wall $= \dfrac{23 \times 6.6^2}{8} - 64.17$

$= 61.1$ kN m/m (tension away from water face)

BM at mid of short wall $= \dfrac{23 \times 4.2^2}{8} - 64.17$

$= -13.5$ kN m/m (tension at water face)

Effective thickness required for walls

$$= \sqrt{\frac{M}{k\,b}} = \sqrt{\frac{64.17 \times 10^6}{1.155 \times 1000}} = 235.7 \text{ mm}$$

Provide overall thickness of wall as 280 mm ($t = 280$ mm)

∴ Effective thickness available,

$d = 280 - 35 = 245$ mm > 235.7 mm (O. K.)

(3) *Pull in Walls*

Direct tension in long wall at 2.3 m from top $= \dfrac{p\,B}{2} = \dfrac{23 \times 4.2}{2} = 48.3$ kN/m

Direct tension in short wall at 2.3 m from top $= \frac{pL}{2} = \frac{23 \times 6.6}{2} = 75.9$ kN/m

(4) *Horizontal Reinforcement in Long Walls*

Maximum horizontal BM,

$M' = 64.2$ kN m/m at corner (tension at water face)

$= 61.1$ kN m/m at corner (tension away from water face)

Maximum pull, $T = 48.3$ kN/m

Corner Section

$$\text{Net BM, } M = M' - Tx \;;\quad x = d - t/2 = 245 - \frac{280}{2} = 105 \text{ mm}$$

$$= 64.2 - 48.3 \times 0.105 = 59.1 \text{ kN m/m}$$

$$\text{Steel for BM, } A_{s1} = \frac{M}{\sigma_{st}\, j\, d} = \frac{59.1 \times 10^6}{150 \times 0.874 \times 245} = 1840 \text{ mm}^2\text{/m}$$

$$\text{Steel for pull, } A_{s2} = \frac{T}{\sigma_{st}} = \frac{48.3 \times 10^3}{150} = 322 \text{ mm}^2\text{/m}$$

Total steel required, $A_s = A_{s1} + A_{s2} = 2162$ mm^2/m

Provide ϕ 20 @ 135 mm c/c at inner face near the corners.

Mid Section

As tension is away from water face and steel is at 245 mm (> 225 mm) from water face, therefore, $\sigma_{st} = 190$ MPa

Net BM, $M = M' - Tx = 61.1 - 48.3 \times 0.105 = 56.0$ kN m/m

$$\text{Steel from BM, } A_{s1} = \frac{M}{\sigma_{st}\, j\, d} = \frac{56.0 \times 10^6}{190 \times 0.892 \times 245} = 1349 \text{ mm}^2\text{/m}$$

Steel for pull, $A_{s2} = 322$ mm^2/m

Total steel required, $A_s = A_{s1} + A_{s2} = 1671$ mm^2/m

Provide ϕ 20 @ 135 mm c/c at outer face in the middle.

The reinforcement details are shown in Figure 22.10.

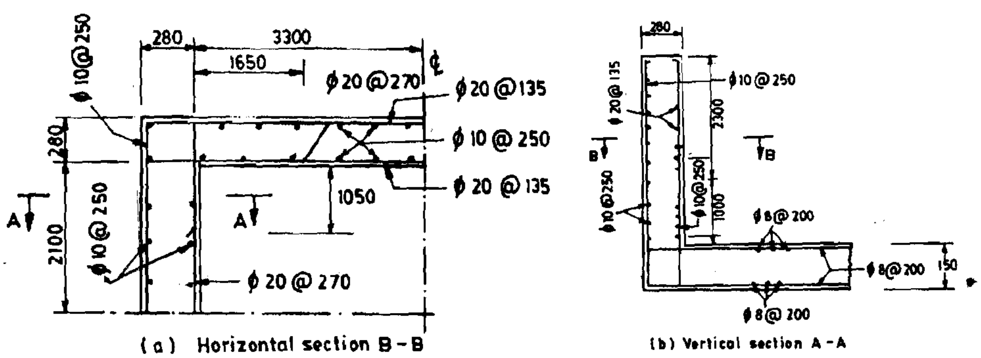


Figure 22.10 : Reinforcement Detail in Rectangular Tank

(5) *Horizontal Reinforcement in Short Walls*

Maximum horizontal BM,

$M' = 64.2$ kN m/m at corner (tension at water face)

$= 13.5$ kN m/m at mid (tension at water face)

Maximum pull, $T = 75.9$ kN/m

Corner Section

Net BM, $M = M' - Tx = 64.2 - 75.9 \times 0.105 = 56.2$ kN m/m

$$\text{Steel for BM, } A_{s1} = \frac{M}{\sigma_{st}\, j\, d} = \frac{56.2 \times 10^6}{150 \times 0.874 \times 245} = 1750 \text{ mm}^2\text{/m}$$

$$\text{Steel for pull, } A_{s2} = \frac{T}{\sigma_{st}} = \frac{75.9 \times 10^6}{150} = 506 \text{ mm}^2\text{/m}$$

$$\text{Total steel, } A_s = A_{s1} + A_{s2} = 2256 \text{ mm}^2\text{/m}$$

Provide ϕ 20 @ 135 mm c/c at the inner face near the corner in short wall.

Mid Section

BM is small, only nominal steel will be enough.

(6) *Vertical Reinforcement in Walls*

$$\text{Maximum cantilever BM, } M = \gamma H \frac{h^2}{6} = 10 \times 3.3 \times \frac{1^2}{6} = 5.5 \text{ kN m/m}$$

$$\text{Area of steel required, } A_s = \frac{5.5 \times 10^6}{150 \times 0.874 \times 245} = 172 \text{ mm}^2\text{/m}$$

$$\text{Minimum percentage of steel} = 0.24 - \left(\frac{280 - 100}{350}\right) \times 0.08 = 0.20$$

$$\text{Minimum Area of steel, } A_{sm} = \frac{0.2}{100} \times 280 \times 1000 = 560 \text{ mm}^2$$

Provide ϕ 10 @ 250 mm c/c on both faces.

(7) *Base Slab*

As the tank rests on ground, provide 150 mm thick slab and provide ϕ 8 @ 200 mm c/c both ways at top and bottom.

The reinforcement details are shown in Figure 22.10.

SAQ 3

How is the design of circular tank with flexible base different from the design of circular tank with rigid base?

SAQ 4

How does the L/B ratio affect the design of criteria of a rectangulra tank?

22.4.2 Overhead Tanks

Cylindrical water tanks are commonly employed in overhead water tanks. A circular cylinder when subjected to a radial force will develop hoop force in the shell which will be tensile if force is acting from inside and compressive if acting from outside. The bending moments in the shell are generated either due to boundary effects or due to unsymmetrical loads. A proper selection of boundary connections can minimise the bending moments. Some of the typical examples of commonly employed overhead water tanks are shown in Figure 22.11. The tanks shown in Figure 22.11(a) and (b) can be used for small capacity of upto 100 kl. The roof and the bottom slab are of flat slab or of beam

100 to 700 kl and those in Figure 22.11(f) and (g) can be used for capacities upto 3000 kl. For large capacity tanks with capacity higher than 2500 kl, the tanks shown in Figure 22.12 having large number of columns can be used. The tank shown in Figure 22.11(f) is known as Intze tank which is very popular because of its dominant membrane action. Its design procedure has been discussed in detail in the subsequent section.

Intze Tank

The Intze tank has got structural as well as architectural advantages over other shapes by making the better use of circular shapes. These tanks are commonly employed for capacities ranging from 100 to 3000 kl. Figure 22.13 shows half section a typical Intze tank whose main structural components are :

(a) Roof Dome

(b) Top Ring Beam

(c) Vertical Wall

(d) Middle Ring Beam

(e) Conical Dome

(f) Bottom Dome

(g) Bottom Ring Beam

(h) Staging (column and brace system or cylindrical shaft)

(i) Foundations

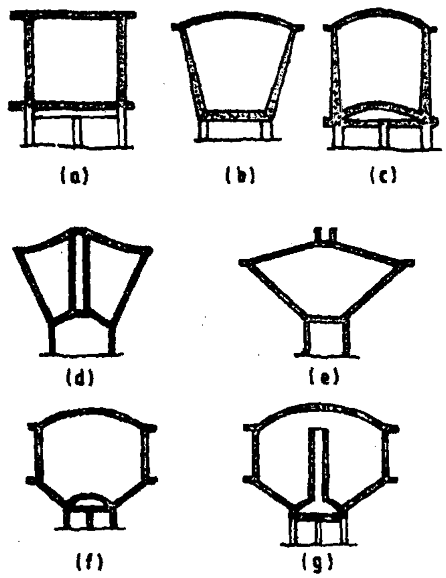


Figure 22.11 : Typical Shapes of Overhead Water Tanks

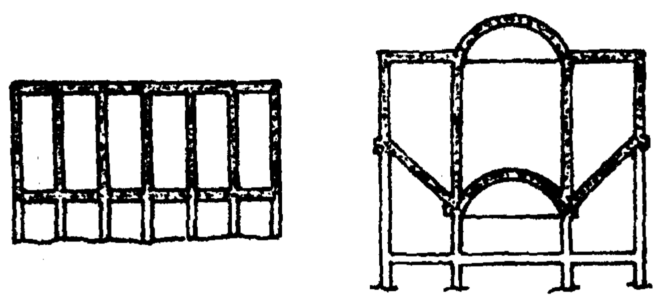


Figure 22.12 : Multiple Columns in Water Tank

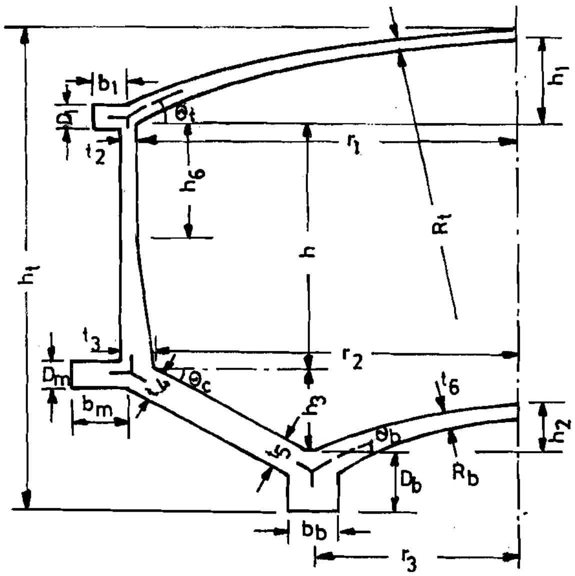


Figure 22.13 : Intze Tank Notations

The proportioning of the tank for known capacity is done by first assuming the inside radius of cylindrical wall at top r_1. The rise of the top dome can be selected in the range of $r_1/6$ to $r_1/3$ depending on the size of the tank. The free board height h_6 is normally taken to vary from 150 to 250 mm depending upon the capacity of the tank. The expression used for the calculation of net capacity of the tank is

$$Q = Q_1 + Q_2 - Q_3$$

where, Q_1 = volume of liquid in the cylindrical portion of tank.

$$= \frac{\pi}{2}\left(r_1^2 + r_2^2\right)\left(h - h_6\right)$$

Q_2 = Volume of a frustum of the cone

$$= \frac{\pi}{3}\left(r_2^2 + r_2\, r_3 + r_3^2\right) h_3$$

Q_3 = Volume of a segment of the bottom spherical dome

$$= \frac{\pi}{6}\left(3\, r_3^2 + h_2^2\right) h_2$$

r_1 = Inside radius of the wall at top

r_2 = Inside radius at the base of the wall

r_3 = Radius of bottom ring beam

h = Height of wall

h_1 = Rise of the top dome

h_2 = Rise of the bottom dome

h_3 = Rise of the cone

h_6 = Free board

$$\therefore\ Q = \frac{\pi}{6}\left[3\left(r_1^2 + r_2^2\right)\left(h - h_6\right) + 2\left(r_2^2 + r_2\, r_3 + r_3^2\right) h_3 - \left(3\, r_3^2 + h_2^2\right) h_2\right]$$

For known capacity, one can assume r_2 ($\approx 0.99\, r_1$), r_3 ($\approx 0.5\, r_1$), h_2 and h_3 and then determine the height h of the tank. The slope of conical wall is taken approximately as 45°.

Membrane analysis of each of the elements is normally employed in the design of the tank. The membrane theory is an equilibrium method, therefore, though the tank may be structurally strong but can lead to cracking and leakage at the junction of the elements because of the unaccounted stresses introduced due to the ignoring of the compatibility of displacement at joints. It is, therefore, recommended that either the compatibility of displacement be satisfied at the joints or nominal additional reinforcement be provided in the vicinity of joints. The procedure of design has been clearly explained through Example 22.4.

Example 22.4

Design an overhead water tank for a capacity of 700 kl with 17.5 m staging. The net safe bearing capacity of soil at 2 m depth is 100 kN/m^2. The tank is located in Mumbai.

Solution

(1) *Data and Basic Dimensions*

Type of tank = Intze tank

Capacity = 700 kl

Staging = 17.5 m

Net safe bearing capacity at 2 m depth = 100 kN/m^2

Depth of foundation = 2.0 m

Type of staging = cylindrical shaft (assumed)

Concrete : M 15 for foundation, staircase, and top dome

M 20 for shaft and the tank

Steel : HYSD with $f_y = 415$ MPa

(2) *Inside Surface Dimensions of the Tank*

Some of the dimensions of the tank are assumed as below :

Radius of the vertical cylinderical wall at top, $r_1 = 6.30$ m

Radius of the vertical cylinderical wall at base $r_2 = 6.15$ m

Rise of the top dome, $h_1 = 1.50$ m

Rise of the bottom dome $h_2 = 1.70$ m

Centre to centre radius of staging, $r_3 = 4.40$ m

Rise of conical shell, $h_3 = 1.75$ m

Free board = 0.15 m

(3) *Capacity Calculation*

Total volume of water is,

$$V = \pi\left[\frac{1}{2}\left(r_1^2 + r_2^2\right)(h - 0.15) + \left(r_2^2 + r_2\, r_3 + r_3^2\right)\frac{h_3}{3} - \left(3\, r_3^2 + h_2^2\right)\frac{h_2}{6}\right] = 700 \text{ m}^3$$

or,

$$\pi\left[\frac{1}{2}\left(6.3^2 + 6.15^2\right)(h - 0.15) + \left(6.15^2 + 6.15 \times 4.4 + 4.4^2\right) \times \frac{1.75}{3} - \left(3 \times 4.4^2 + 1.7^2\right)\right.$$

or, $121.756\,(h - 0.15) + 100.112 = 700$

or, $h = 5.08$ m

Providing $h = 5.10$ m

(4) *Allowable Stresses and other Code Specifications*

M 15 grade concrete :

Permissible bending compressive stress, $\sigma_{cbc} = 5$ MPa

Permissible direct compressive stress, $\sigma_{cc} = 4$ MPa

M 20 grade concrete (water retaining) :

Permissible direct tensile stress, $\sigma_{ct} = 1.2$ MPa

Permissible bending tensile stress, σ_{cbt} = 1.7 MPa

Fe 415 HYSD steel :

Permissible tensile stress, σ_{st} = 230 MPa in ordinary RCC

= 150 MPa upto 225 mm from water face

= 190 MPa beyond 225 mm from water face

Permissible stresses will be increased by 33.33% for wind or earthquake load condition.

Seismic coefficients :

Soil structure interaction for raft in soft soils, $\beta = 1.0$

Importance factor, $I = 1.5$

seismic zone factor, $F_o = 0.2$

Damping = 5%

Basic wind pressure = 1.2 kN/m^2

(5) *Design of Roof Dome*

Let the thickness of dome, $t = 0.075$ m

Rise of the dome, $h_1 = 1.5$ m

Chord radius, $r_1 = 6.3$ m

Surface radius, $$R = \frac{1}{2}\left(\frac{r_1^2}{h_1} + h_1\right)$$

$$= \frac{1}{2}\left(\frac{6.3^2}{1.5} + 1.5\right) = 13.98 \text{ m}$$

$$\cos\phi = 1 - \frac{1.5}{13.98} = 0.893, \text{ or } \phi = 26.785^{\circ}$$

Self weight of dome, $W_d = 0.075 \times 24 = 1.80$ kN/m^2

Live load, $W_l = 0.75$ kN/m^2

Total load, $W = W_d + W_l = 2.55$ kN/m^2

Total live load on the dome, $W_1 = 2\pi R h_1 W_l$

$$= 2\pi \times 13.98 \times 1.5 \times 0.75 = 98.8 \text{ kN}$$

Self weight of the dome, $W_2 = 2\pi\ r h_1,\ Wd$

$$= 2\pi \times 13.98 \times 1.5 \times 1.8$$

$$= 237.2 \text{ kN}$$

Maximum meridional thrust (at the edges),

$$N_\phi = \frac{wR}{1 + \cos\phi} = \frac{2.55 \times 13.98}{1 + 0.893} = 18.83 \text{ kN/m}$$

Meridional stress, $\frac{N_\phi}{t} = \frac{18.83 \times 10^{-3}}{0.075} = 0.251$ Mpa (compressive)

Hoop stress is also compressive over the entire dome and it is less than the meridional stress. The actual compressive stress is far less than allowable compressive stress, therefore, only nominal rienforcement will be provided.

Minimum reinforcement required in either direction

$$= \frac{0.15}{100} \times 1000 \times 75 = 112.5 \text{ mm}^2\text{/m}$$

Provide ϕ 8 @ 225 mm c/c bothways ($\therefore$ spacing $\not>$ 3 times the thickness of dome)

Area of steel provided $= \frac{1000}{225} \times \frac{\pi}{4} \times 8^2 = 223$ mm^2/m (0.297%)

(6) *Design of Top Ring Beam (M 20 concrete)*

Horizontal component of meridional thrust for the top dome will cause hoop tension in the top ring beam.

Hoop tension in ring beam, $T = (N_p \cos \phi)\, r_1$

$$= 18.83 \times 0.893 \times 6.3 = 105.94 \text{ kN}$$

Area of steel required, $A_s = \frac{T}{\sigma_{st}} = \frac{105.94 \times 10^3}{150} = 706.24 \text{ mm}^2$

Provide 4 ϕ 16, A_s provided = 804 mm^2

Assuming the width of beam, b_1 as 250 mm.

Let D_1 be its depth.

Tensile stress in concrete $= \frac{T}{b_1 D_1 + (m-1) A_s} \leq \sigma_{ct}$

or, $\frac{105.94 \times 10^3}{250\, D_1 + (19-1) \times 804} \leq 1.2$ or, $D_1 \geq 295.25$ mm = 300 mm (say)

Keep the size of beam as 250 $\times$ 300 mm.

Provide ϕ 6 ties @ 250 mm c/c.

Weight of the ring beam, $W_3 = 2\pi \left(r_1 + \frac{b_1}{2} \right) b_1 D_1 \gamma_c$

$$= 2\pi \left(6.3 + \frac{0.25}{2} \right) 0.25 \times 0.3 \times 25 = 75.7 \text{ kN}$$

(7) *Design of Tank Wall*

Height of wall, $h = 5.1$ m

Hoop tension at base of the wall, $T_1 = \gamma\, r_2\, h = 10 \times 6.15 \times 5.1 = 313.65$ kN

Hoop tension at 4.1 m from top, $T_2 = 10 \times 6.15 \times 4.1 = 252.15$ kN

Total hoop tension in the bottom 1 m depth, $T = \frac{1}{2}\left(T_1 + T_2\right) = 282.90$ kN

Area of steel required in the bottom 1 m depth,

$$A_s = \frac{T}{\sigma_{st}} = \frac{282.90 \times 10^3}{150} = 1886 \text{ mm}^2\text{/m}$$

Provide 9 ϕ 12 on each face, A_s (provided) = 2034 mm^2

Tensile stress in concrete $= \frac{T}{A_c + (m-1) A_s} \leq \sigma_{ct}$

or, $\frac{282.90 \times 10^3}{A_c + (13-1) \times 2034} = 1.2$ or, $A_c \geq 211342$ mm^2

Thickness required = 211.34 mm

Providing wall 230 mm (say t_3) thick at base and tapered to 100 mm (say t_2) at the top.

Average thickness between 4.1 and 5.1 m depth

$$= 100 + \left(\frac{230 - 100}{5.1} \right) \times 4.6 = 217.25 \text{ mm} > 211.34 \text{ mm} \quad \text{(O.K.)}$$

The circumferential reinforcement in vertical wall is given in Table 22.11.

Table 22.11 : Cicumferential Reinforcement in Vertical Wall

Distance from Top (m)	Hoop Tension T (kN)	Reinforcement	
		Required (mm^2)	Number of ϕ 12 Bars
0.0 to 1.1	33.8	226	4 (middle)
1.1 to 2.1	98.4	656	6 (middle)
2.1 to 3.1	159.9	1066	5 (each face)
3.1 to 4.1	221.4	1476	7 (each face)
4.1 to 5.1	282.9	1886	9 (each face)

$$\text{Weight of the wall, } W_u = 2\pi \frac{(r_1 + r_2)}{2} \cdot \left(\frac{t_2 + t_3}{2}\right) (h - D_1)\gamma_c$$

$$= \frac{\pi}{2} (6.30 + 6.15)\ (0.23 + 0.10)\ (5.1 - 0.3) \times 25$$

$$= 774.4 \text{ kN}$$

Average thickness of wall = 165 mm

Minimum percentage of HYSD steel for 165 mm thickness

$$= 0.24 - \left(\frac{165 - 100}{350}\right) \times 0.08 = 0.225$$

$$\text{Minimum vertical steel required} = \frac{0.225}{100} \times 165 \times 1000 = 372 \text{ mm}^2\text{/m}$$

Provide ϕ 8 @ 250 mm c/c (vertical) on both faces,

A_s (provided) = 400 mm^2/m

(8) *Design of Middle Walking Gallery*

Width of gallery = 1 m

The design live load is taken as 1.5 kN/m^2 or 1 kN load placed near the tip of the balcony (say about 75 mm from the free end of the balcony).

Assuming the width of middle ring beam as 500 mm. This also serves partly as a balcony at this height. So, the cantilever span of the slab = 0.5 m

Self weight of slab (75 mm thick) = $0.075 \times 0.5 \times 25 = 0.94$ kN/m

Railing load at 75 mm from tip = 0.80 kN/m

BM due to self weight and railing

$$= 0.94 \times \frac{0.5}{2} + 0.8\ (0.5 - 0.075) = 0.575 \text{ kN m/m}$$

$$\text{BM due to LL (Udl)} = 1.5 \times \frac{0.5^2}{2} = 0.188 \text{ kN m/m}$$

BM due to LL (point load) = $1 \times (0.5 - 0.075) = 0.425$ kN m/m

Maximum design BM, $M = 0.575 + 0.425 = 1.0$ kN m/m

$$\text{Area of steel required, } A_s = \frac{M}{\sigma_{st}\, j\, d} = \frac{1.0 \times 10^6}{230 \times 0.906 \times 45} = 107 \text{ mm}^2$$

Provide ϕ 8 @ 250 mm c/c radial bars and anchored into the ring beam.

Provide 2 ϕ 8 in circumferential direction in the 500 mm projection.

Total weight of the slab, $W_5 = 2\pi\ (6.15 + 0.23 + 0.5) \times 0.5 \times 0.075 \times 25$

$= 40.5$ kN

Total live load on the balcony,

$$W_6 = 2\pi\ (6.15 + 0.23 + 0.5) \times 1 \times 1.5 = 64.8 \text{ kN}$$

Total weight of railing, $W_7 = 2\pi \times (6.15 + 0.23 + 1.0) \times 0.8 = 37.1$ kN

(9) *Design of Middle Ring Beam*

Self weight $= 0.5 \times 0.5 \times 25 = 6.25$ kN/m

Total self weight, $W_8 = 2\pi\ (6.15 + 0.23 + 0.25) \times 6.25 = 260.4$ kN

LL from the top dome, $W_1 = 98.8$ kN

Weight of the top dome, $W_2 = 237.2$ kN

Weight of the top ring beam $W_3 = 75.7$ kN

Weight of the vertical wall, $W_4 = 774.4$ kN

Weight of the gallery slab, $W_5 = 40.5$ kN

LL on the gallery, $W_6 = 64.8$ kN

Weight of the railing, $W_7 = 37.1$ kN

Weight of water on the tapered face of wall,

$$W_9 = \pi\ (6.3^2 - 6.15^2) \times 5.1 \times \frac{10}{2} = 149.6 \text{ kN}$$

Total load transfered to the conical shell, $W_{10} = W_1 + \ldots + W_9$

$= 1738.5$ kN

$$\text{Hoop tension in the ring beam, } T = \frac{W_{10} \cot\theta}{2\pi} = \frac{1738.5 \cot 45^\circ}{2\pi} = 276.7 \text{ kN}$$

$$\text{Steel required in the ring beam, } A_s = \frac{T}{\sigma_{st}} = \frac{276.7 \times 10^3}{150} = 1845 \text{ mm}^2$$

Provide 10 ϕ 16, A_s (provided) $= 10 \times 201 = 2010$ mm^2

$$\text{Tensile stress in the ring beam} = \frac{T}{A_c + (m-1) A_s} \leq \sigma_{ct}$$

$$\text{or,} \quad \frac{276.7 \times 10^3}{A_c + (13-1) \times 2010} \leq 1.2 \quad \text{or,} \quad A_c = 206464 \text{ mm}^2$$

Provide 500×450 mm ring beam and ϕ 8 @ 450 mm c/c

Actual weight of ring beam, $W_8 = 234.4$ kN

Revised total load, $W_{10} = 1738.5 - 260.4 + 234.4 = 1712.5$ kN

(10) *Design of Conical Shell*

There is meridional stress and hoop stress in the conical shell. The meridional thrust is maximum at the base of the cone whereas hoop tension varies along the depth. Assuming the average thickness of conical slab as 450 mm.

Height of the cone, $h_3 = 1.75$ mm

Slope of wall, $\theta = 45^\circ$

Self weight of slab (450 mm) $= 0.45 \times 25 = 11.25$ kN/m^2

Length of slab, $L = 1.75\sqrt{2} = 2.475$ m

$$\text{Weight of the conical wall, } W_{11} = 2\pi\gamma\,\frac{(r_2 + r_3)}{2}\,t\,L$$

$$= \pi \times 25 \times (6.15 + 4.4) \times 0.45 \times 2.475$$

$$= 922.8 \text{ kN}$$

Weight of the water over the conical shell,

$$W_{12} = \pi\gamma\left(r_2^2 - r_3^2\right)(h + 0.5\,h_3)$$
$$= \pi \times 10 \times \left(6.15^2 - 4.4^2\right) \times (5.1 + 0.5 \times 1.75)$$
$$= 3465.6 \text{ kN}$$

Total load on the conical shell at its base, $W_{13} = W_{10} + W_{11} + W_{12}$
$= 6100.9$ kN

Meridional thrust at the base of conical shell,

$$N = \frac{W_{13}\,\text{cosec}\,\theta}{2\,\pi\,r_3} = \frac{6100.9\,\text{cosec}\,45^o}{2\,\pi \times 4.4} = 312.1 \text{ kN/m}$$

Horizontal component of the thrust,

$$H_1 = N\cos\theta = 312.1\cos 45^o = 220.7 \text{ kN/m}$$

Assuming the thickness of conical shell at the base as 400 mm.

Meridional stress at the base of conical shell

$$= \frac{312.1 \times 10^3}{1000 \times 400} = 0.78 \text{ MPa (small)}$$

Thickness of the shell will be governed by the hoop tension.

Radius of cone at height y from base, $r_y = (4.4 + y)$ m

Height of water at this level,

$$h_y = h + (h_3 - y) = 5.1 + (1.75 - y) = (6.85 - y) \text{ m}$$

Normal load on the slanting slab, p_y = water pressure + component of the weight of slab

$$= (6.85 - y) + 11.25\cos 45^o$$
$$= 76.45 - 10\,y$$

Hoop tension in the slab,

$$T = (p_y\,\text{cosec}\,\theta)\,r_y = (76.45 - 10\,y)\ \text{cosec}\,45^o\ (4.4 + y)$$
$$= \sqrt{2}\ (76.45 - 10\,y)\ (4.4 + y)$$
$$= 475.7 \text{ kN/m at } y = 0 \text{ (bottom ring beam)}$$
$$= 505.0 \text{ kN/m at } y = 0.875 \text{ m}$$
$$= 512.7 \text{ kN/m at } y = 1.75 \text{ m (middle ring beam)}$$

Total hoop tension in the slab, $T = \frac{1}{2}\,(T_1 + 2\,T_2 + T_3)\,\frac{L}{2}$

$$= \frac{1}{4}\,(475.5 + 2 \times 505.0 + 512.7) \times 2.475$$
$$= 1236.5 \text{ kN}$$

Total area of hoop tension required, $A_s = \frac{1236.5 \times 10^3}{150} = 8244 \text{ mm}^2$

Provide 20 ϕ 16 at the inner face and 22 ϕ 16 at the outer face of the slab.

Total steep provided, $A_s = 42 \times 201 = 8442 \text{ mm}^2$

Tensile stress in concrete $= \frac{T}{A_c + (m-1)\,A_s} \leq \sigma_{ct}$

or, $\frac{1236.5 \times 10^3}{A_c + (13-1) \times 8442} \leq 1.2$ or, $A_c \geq 929113 \text{ mm}^2$

Average thickness of slab, $t = \frac{A_c}{L} = \frac{929113}{2475} = 375.4$ mm

Provide thickness of slab at the top as $t_4 = 430$ mm

and at base as $t_5 = 350$ mm

Average thickness of slab = 390 mm > 375.4 mm (O. K.)

Revised weight of conical shell, $W_{11} = 922.8 \times \frac{0.39}{0.45} = 799.8$ kN

Total load on the conical shell at its base $W_{13} = W_{10} + W_{11} + W_{12} = 5977.9$ kN

Minimum percentage of steel in conical shell

$$= 0.24 - \frac{(0.24 - 0.16)}{(450 - 100)} \times (390 - 100)$$

$$= 0.174$$

Minimum steel = $\frac{0.174}{100} \times 1000 \times 390 = 678 \text{ mm}^2/\text{m}$

Provide ϕ 12 @ 300 mm c/c in radial direction on each surface

A_s (provided) = $2 \times \frac{1000}{300} \times 113 = 753 \text{ mm}^2/\text{m}$

(11) *Design of Bottom Dome*

Half chord length, $r_3 = 4.4$ m

Rise of the dome, $h_2 = 1.7$ m

Thickness of dome, $t_6 = 0.15$ m (say)

Radius of the dome, $R = \frac{1}{z\,h_2}\left(r_3^2 + h_2^2\right) = \frac{1}{2 \times 1.7}\left(4.4^2 + 1.7^2\right) = 6.544$ m

Self weight of the dome,

$$W_{14} = 2\,\pi\,R\,h_2\,t_6\,\gamma_c = 2\,\pi \times 6.544 \times 1.7 \times 0.15 \times 25$$

$$= 262.1 \text{ kN}$$

Weight of the water on the dome,

$$W_{15} = \pi\,\gamma\left[r_3^2\,(h + h_3) - (3\,r_3^2 + h_2^2)\,\frac{h_2}{6}\right]$$

$$= 10\,\pi\left[4.4^2\,(5.1 + 1.75) - (3 \times 4.4^2 + 1.7^2) \times \frac{1.7}{6}\right]$$

$$= 3623.5 \text{ kN}$$

Totel weight on the dome, $W_{16} = W_{14} + W_{15} = 262.1 + 3623.5$

$$= 3885.6 \text{ kN}$$

Semicentral angle, $\sin\phi = \frac{r_3}{R} = \frac{4.4}{6.544} = 0.6724$

Meridional thrust,

$$N_\phi = \frac{W_{16}\,R}{r\,\pi\,r_3^2} = \frac{3885.6 \times 6.544}{2\,\pi \times 4.4^2} = 209.0 \text{ kN/m} \quad \text{(compressive)}$$

Meridional stress = $\frac{N_\phi}{b\,t_6} = \frac{209.0 \times 10^3}{1000 \times 150} = 1.393$ MPa (small)

Hoop compression is also low, therefore, provide only minimum steel.

Minimum percentage of steel = $0.24 - \frac{0.08}{350} \times 50 = 0.23$

Minimum steel, $A_s = \frac{0.23}{100} \times 1000 \times 150 = 345 \text{ mm}^2/\text{m}$

Provide ϕ 12 @ 300 mm c/c both ways

Horizontal thurst, $H_2 = N_\phi \cos\phi = 209.0 \left(\frac{6.544 - 1.7}{6.544}\right)$

$= 154.7$ kN/m

(12) *Design of Bottom Ring Beam*

Horizontal thrust on ring beam from conical shell, $H_1 = 220.7$ kN/m

Horizontal thrust on ring beam from bottom dome $H_2 = 154.5$ kN/m

Net thrust on the ring beam, $H_3 = H_1 - H_2 = 66.0$ kN/m (inward)

Hoop compression, $T = H_3 r_3 = 66.0 \times 4.4 = 290.4$ kN

Provide a ring beam of section 400×400 mm

Compressive stress $= \dfrac{290.4 \times 10^3}{400 \times 400} = 1.815$ MPa (small)

Minimum steel (0.24%) $= \dfrac{0.24}{100} \times 400 \times 400 = 384\ \text{mm}^2$

Provide 4 ϕ 16 and ϕ 8 ties @ 300 mm c/c

Weight of the beam, $W_{17} = 2\pi \times 4.4 \times 0.4 \times 0.4 \times 25$

$= 110.6$ kN

(13) *Design of Cylindrical Shaft*

Staging of the tank, $h = 17.5$ m = Height of shaft above ground

Radius of the centre line of the shaft, $r_3 = 4.4$ m

Depth of foundation = 2.0 m

Assuming the thickness of the shaft wall, t as 100 mm

The thickness of shaft is assumed as 300 mm at the top of the foundation slab which is tapered to 100 mm at 1 m below GL and then remains constant upto the top. The loads acting at 1 m below GL on the shaft are listed below.

Weight of shaft upto 1 m below GL, $W_{18} = 2\pi r_3 t h_5 \gamma_c$

$= 2\pi \times 4.4 \times 0.1 \times (17.5 + 1 - 0.4) \times 25 = 1251.0$ kN

Weight of the bottom ring beam, $W_{17} = 110.6$ kN

Weight of the bottom dome, $W_{14} = 262.1$ kN

Weight of the conical shell, $W_{11} = 799.8$ kN

Weight of the middle ring beam, $W_8 = 234.4$ kN

Weight of the gallery slab, $W_5 = 40.5$ kN

Weight of the tank's vertical wall, $W_4 = 774.4$ kN

Weight of the top ring beam, $W_3 = 75.7$ kN

Weight of the top dome, $W_2 = 237.2$ kN

Total weight of concrete upto 1 m below GL, $W_{20} = 3785.7$ kN

Live load $= W_1 + W_6 = 98.8 + 64.8 = 163.6$ kN

Weight of railing, $W_7 = 37.1$ kN

Weight of water = 7000.0 kN

Total water, live and railing load, $W_{21} = 7200.7$ kN

Weight of staircase $\approx 1 \times 0.3 \times 0.2 \times \dfrac{18.5}{0.3} \times 25 = 92.5$ kN

Staircase's railing and partial LL etc. = 160.0 kN

Total load from staircase = 92.5 + 160.0 = 252.5 kN

Total load at the base of the shaft,

$$W_{22} = 3785.7 + 7200.7 + 252.5 = 11238.9 \text{ kN}$$

Area of cross-section of shaft, $A = 2\pi r_3 t = 2\pi \times 4.4 \times 0.1 = 2.765 \text{ m}^2$

Moment of inertia of the section, $I = \pi r_3^3 t = \pi \times 4.4^3 \times 0.1 = 26.761 \text{ m}^4$

Axial stress on the shaft wall $= \dfrac{W_{22}}{A} = \dfrac{11238.9 \times 10^3}{2.765 \times 10^6} = 4.07 \text{ MPa}$

The shaft is a cylindrical shell subjected to axial force and bending.

Allowable buckling compressive stress of the shell,

$$\sigma_{cr} = \frac{0.25 f_{ck}}{1 + f_{ck}/F_{cr}}$$

where, $F_{cr} = \dfrac{0.2\,E\,t}{R} = \dfrac{0.2 \times 5700\sqrt{20} \times 0.1}{4.4} = 115.9 \text{ MPa}$

$f_{ck} = 1.2 \times 20 = 24 \text{ MPa}$ (including age effect)

$$\sigma_{cr} = \frac{0.25 \times 24}{1 + \dfrac{24}{115.9}} = 4.97 \text{ MPa} > 4.07 \text{ MPa}$$

Therefore, only nominal steel is required.

Minimum steel in shaft, $A_s = \dfrac{0.24}{100} \times 1000 \times 100 = 240 \text{ mm}^2\text{/m}$

Provide ϕ 8 @ 200 mm c/c in longitudinal as well as circumferential direction.

$$A_s \text{ (provided)} = \frac{1000}{200} \times 50 = 250 \text{ mm}^2\text{/m}$$

Wind Loads :

Basic wind pressure, $p = 0.6\ V_z^2 \text{ N/m}^2$

where, Design wind speed,

$V_z = V_b K_1 K_2 K_3 = 44 \times 1 \times 1.1 \times 1 = 48.4 \text{ m/s}$

$\therefore\ p = 0.6 \times 48.4^2 = 1406 \text{ N/m}^2 = 1.4 \text{ kN/m}^2$

V_b = basic wind speed

= 44 m/s for Mumbai

K_1 = Risk coefficient = 1.0

K_2 = Terrain and height factor = 1.1

K_3 = Topography factor = 1.0 for slope within $\pm 3^\circ$

Vertically projected area of the tank,

$$A_1 = \frac{2}{3}[(6.3 + 0.1) \times 2 \times (1.5 + 0.075)]$$

$$+ (6.3 +).1) \times 2 \times 5.1$$

$$+ \frac{2}{3}(6.3 + 0.1) \times 2 \times (1.7 + 0.4)$$

$$= 12.8\left[\frac{2}{3} \times 1.575 + 5.1 + \frac{2}{3} \times 2.1\right] = 96.6 \text{ m}^2$$

Vertically projected area of shaft, $A_2 = 2\left(4.4 + \dfrac{0.1}{2}\right)(17.5 - 0.4) = 152.2 \text{ m}^2$

Shape factor = 0.7

Maximum BM at 1 m below GL due to mid

$$= 0.7\,p\left[(1 + 17.5 + 1.75 + 2.55)\ A_1 + \frac{1}{2}\ (1 + 17.1)\ A_2\right]$$

$$= 0.7 \times 1.3\ [22.8 \times 96.6 + 9.05 \times 152.2] = 3257.7 \text{ kNm}$$

Extra BM for gallery and railing effects = 60 kNm (say)

Total maximum BM, $M_w = 3257.7 + 60 = 3317.7$ kNm

Maximum SF due to wind = $0.7\,p\,\left(A_1 + A_2\right)$

$$= 0.7 \times 1.4\ (96.6 + 152.2) = 243.8 \text{ kN}$$

Extra SF for gallery and railing effects = 12 kN (say)

Total maximum SF, $V_w = 243.8 + 12 = 255.8$ kN

Seismic Loads :

Weight of the tank including water and 25% of the LL,

$$W_{23} = \left(W_{20} - W_{18}\right) + W_7 + 7000 + 0.25 \times 163.6$$

$$= (3785.7 - 1251.0) + 37.1 + 7000 + 0.25 \times 163.6$$

$$= 9612.7 \text{ kN}$$

Weight of the shaft including the staircase, $W_{24} = W_{18} + 252.5$

$$= 1251.0 + 252.5 = 1503.5 \text{ kN}$$

Effective equivalent load, $W_{25} = W_{23} + \frac{1}{3}\ W_{24} = 9612.7 + \frac{1}{3} \times 1503.5$

$$= 10113.8 \text{ kN}$$

Lateral deflection of top of the tank due to equivalent load,

$$\delta = \frac{W_{25}\,L^3}{3\,EI}\left[1 + \frac{3 \times 0.5\,h}{2\,L}\right]$$

where,

$$L = 17.5 + 1.75 + \frac{5.1}{2} = 21.8 \text{ m}$$

$$E = 5700\sqrt{20} = 25491.2 \text{ MPa} = 25.5 \text{ GPa}$$

$$\frac{1.5\,h}{2\,L} = \frac{1.5 \times 5.1}{2 \times 21.8} = 0.1755$$

$$I = 26.761 \text{ m}^4$$

$$\therefore\ \delta = \frac{10113.8 \times 21.8^3}{3 \times 25.5 \times 10^6 \times 26.761}\ (1 + 0.1755) = 0.060 \text{ m}$$

Natural time period of the tank, $T = 2\pi\sqrt{\frac{\delta}{\gamma}} = 2\pi\sqrt{\frac{0.060}{9.81}} = 0.49$ sec

Average acceleration coefficient for $T = 0.49$ sec and 5% damping,

$$\frac{S_a}{g} = 0.17$$

Horizontal seismic coefficient,

$$\alpha_h = \beta\,I\,F_o\,\frac{S_a}{g} = 1 \times 1.5 \times 0.2 \times 0.17 = 0.051$$

($F_o = 0.2$ for seismic zone III)

BM at 1 m below GL due to seismic loads,

$$M_q = \alpha_h\left[W_{23}\left(1 + 17.5 + 1.75 + \frac{5.1}{2}\right) + W_{24}\,(1 + 17.1)\right]$$

$$= 0.051 \times [9612.7 \times 22.8 + 1503.5 \times 18.1]$$

$$= 12565.5 \text{ kNm}$$

$$\approx 13000 \text{ kNm} \text{ considering extra for EP}$$

Maximum SF due to seismic loads, $V_q = 0.051 \times (9612.7 + 1503.5)$

$$= 566.9 \text{ kN}$$

$$\approx 570 \text{ kN (say)}$$

It can be seen that the seismic loads are critical as compared to wind loads.

Axial load for earthquake condition, $P = W_{23} + W_{24} = 9612.7 + 1503.5$

$$= 11116.2 \text{ kN}$$

Stress in concrete at 1 m below GL

$$= \frac{P}{A} \pm \frac{My}{I}$$

$$= \frac{11116.2 \times 10^3}{2.765 \times 10^6} \pm \frac{13000 \times 10^6 \times 4.45 \times 10^3}{26.761 \times 10^{12}}$$

$$= 4.02 \pm 2.16$$

$$= 6.18 \text{ MPa and}$$

$$= 1.86 \text{ MPa } (\Rightarrow \text{No tension})$$

Allowable stress under seismic condition $= 4.97 \times 1.33 = 6.61$ MPa

Maximum compressive stress in concrete is less than the allowable value without considering the steel.

Average shear stress, $\tau_v = \frac{V_q}{A} = \frac{570 \times 10^3}{2.765 \times 10^6} = 0.21$ MPa

This value is far less than the allowable value, therefore, nominal circumferential steel provided is enough.

Reinforcement in the body of the tank is shown in Figure 22.14.

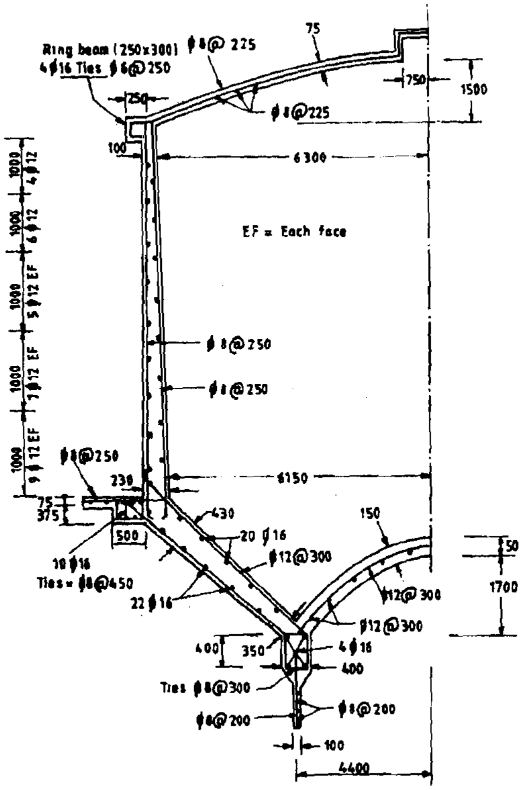


Figure 22.14 : Reinforcement in Intze Tank

22.4.3 Underground Tanks

The tanks required for water-purification or sewage treatment are usually built wholly or partly below ground depending upon the circumstances. In any case, the sides of the tank have to resist earth pressure from outside and water pressure from inside. The worst condition of loading for the design of these tanks are the following :

(i) Empty Tank

(ii) Full Tank

(iii) Testing Condition

(i) **Empty Tank**

When the tank is empty and the soil around is charged with water in a critical condition for outside loading (Figure 22.15(a)).

(ii) **Full Tank**

When the tank is full and the soil around shrinks away from the walls, there will be hydrostatic pressure from inside and no earth pressure (Figure 22.15(b)). This condition can however be avoided by filling the space around with grannular material so that it is always there to give support. Further, the earth pressure, when the tank is full, is passive and is equal to the pressure from inside. Hence the net pressure on the wall is zero. However, for the passive pressure to develop, the wall has to deflect outward to some extent thus causing stresses in it. Hence, it is desirable to also design the walls for a net water pressure of about half the intensity (Figure 22.15(c)).

(iii) **Testing Condition**

Sometimes, the tank is tested for leakage through walls before filling earth around walls. For this condition, the tank is designed for water pressure from inside (Figure 22.15(d)).

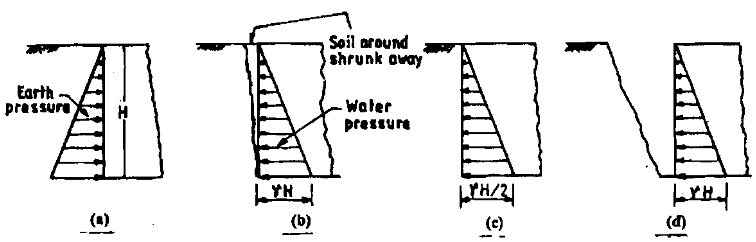


Figure 22.15 : Critical Load Combinations for Underground Water Tank

If the walls of underground water tank are sloped at the angle of repose of the soil, no earth pressure will be exerted on them. They can, then be built with a nominal thickness, to stop leakage of water.

If the subsoil water is likely to rise above the base of the tank, there may be an upward pressure on the base tending to lift it when the tank is empty. In such cases, it will be economical to build the tank partly above ground level so that the base does not have to withstand the water pressure. Otherwise, the base should be extended beyond the walls such that dead load of the tank and earth over the base projection balance the upthrust of water. The base slab should be designed suitably to resist the upthrust of water when the tank is empty.

Example 22.5

Design an underground rectangular water tank of internal dimensions $6.4 \times 3.0 \times 3.0$ m (deep). The tank shall be covered with a roof slab. The soil surrounding the tank remains always dry and its unit weight and angle of repose are 18 kN/m^2 and 28° respectively. The tank is to be tested for leakage before earth is filled around the tank.

Solution

(1) *Data and Design Constants*

The tank with basic dimensions is shown in Figure 22.16.

$L = 6.4$ m, $B = 3.0$ m, $H = 3.0$ m

Capacity with 150 mm free board

$$= 6.4 \times 3.0 \times 2.85 = 54.72 \text{ m}^3 = 54.72 \text{ kl}$$

Concrete grade = M 20

Steel grade = Fe 415 (HYSD)

Unit weight of soil, $\gamma_s = 18 \text{ kN/m}^3$

Angle of repose of soil, $\phi = 28^o$

Permissible direct tensile stress of concrete, $\sigma_{ct} = 1.2$ MPa

Permissible bending tensile stress of concrete, $\sigma_{cbt} = 1.6$ MPa

Permissible bending compressive stress of concrete, $\sigma_{cbc} = 7.0$ MPa

Modular ratio, $m = \dfrac{280}{3\,\sigma_{cbc}} = 13$

Permissible tensile stress of steel,

$$\sigma_{st} = 150 \text{ MPa upto 225 mm from water face}$$

$$= 190 \text{ MPa beyond 225 mm from water face}$$

N. A. depth coefficient, $n = \dfrac{1}{1 + \dfrac{\sigma_{st}}{m\,\sigma_{cbc}}} = 0.378$ for $\sigma_{st} = 150$ MPa

$= 0.324$ for $\sigma_{st} = 190$ MPa

Lever arm coefficient, $j = 1 - n/3 = 0.874$ for $\sigma_{st} = 150$ MPa

$= 0.892$ for $\sigma_{st} = 190$ MPa

$K = \dfrac{1}{2}\sigma_{cbc}\,n\,j = 1.155$ for $\sigma_{st} = 150$ MPa

$= 1.012$ for $\sigma_{st} = 190$ MPa

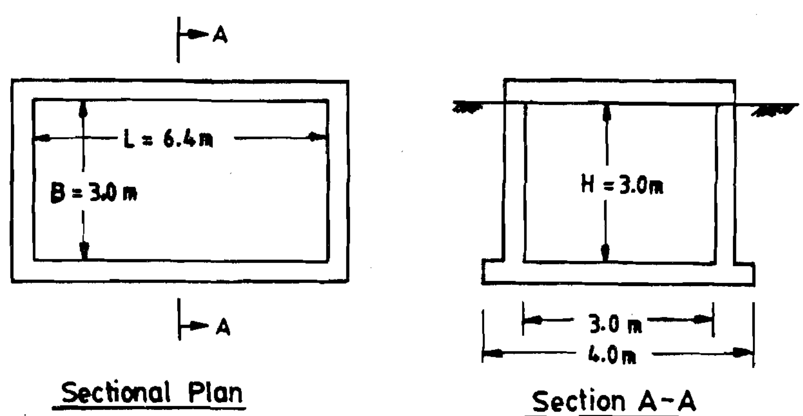


Figure 22.16 : Basic Dimensions of the Tank

(2) *Design of Walls*

There are two critical cases for which the tank walls shall be designed :

(i) Testing condition (Tank full and no surrounding soil)

(ii) Empty Tank Condition

All walls will be designed as propped cantilevers, the prop at the top being provided by the connection of top slab with the walls. The analysis for the two critical cases is presented here under :

(i) *Testing Condition*

As the tank is to be tested for leakage before filling back the surrounding soil, the only pressure acting on the tank walls is water pressure from inside (Figure 22.17).

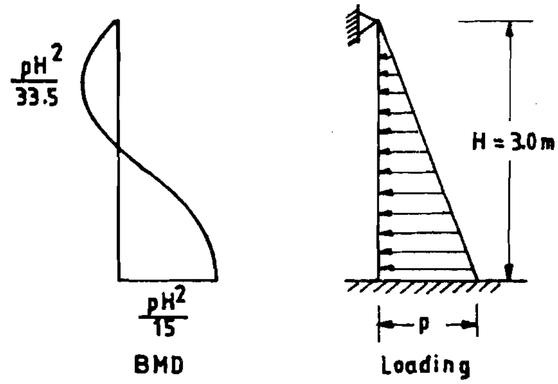


Figure 22.17 : Loading and Bending Moment in Wall

Maximum water pressure, $p = \gamma H = 10 \times 3$

$$= 30 \text{ kN/m}^2$$

BM at base producing tension at water face

$$= \frac{p H^2}{15} = \frac{30 \times 3^2}{15} = 18.0 \text{ kN m/m}$$

Maximum BM causing tension away from water face

$$= \frac{p H^2}{33.5} = \frac{30 \times 3^2}{33.5} = 8.1 \text{ kN m/m}$$

Thickness of wall required for avoiding cracking can be calculated from :

$$\sigma_{cbt} = \frac{1}{6} b D^2 = M$$

or, $\quad 1.6 \times \frac{1}{6} \times 1000 D^2 = 18.0 \times 10^6$

or, $\quad D = 260 \text{ mm}$

Providing 275 mm thick walls ($d = 275 - 40 = 235$ mm)

(ii) *Empty Tank Condition*

When the tank is empty, there will be only active earth pressure acting from outside.

Active earth pressure coefficient,

$$k_a = \frac{1 - \sin \phi}{1 + \sin \phi} = \frac{1 - \sin 28^\circ}{1 + \sin 28^\circ} = 0.361$$

Maximum earth pressure,

$$p = k_a \gamma_s H = 0.361 \times 18 \times 3.0 = 19.5 \text{ kN/m}^2$$

BM at base providing tension away from water face

$$= \frac{p H^2}{15} = \frac{19.5 \times 3^2}{15} = 11.7 \text{ kN m/m}$$

Maximum BM causing tension at water face

$$= \frac{p H^2}{33.5} = \frac{19.5 \times 3^2}{33.5} = 5.2 \text{ kN m/m}$$

(iii) *Area of Steel*

Maximum BM at water face = 18.0 kN m/m

Maximum BM away from water face = 11.7 kN m/m

$$\text{Percentage of minimum steel required} = 0.24 - \frac{175}{350} \times 0.08$$

$$= 0.20$$

Area of minimum steel required,

$$A_{t\,\min} = \frac{0.2}{100} \times 1000 \times 275 = 550\ \text{mm}^2$$

$$= 275\ \text{mm}^2 \text{ on each face}$$

$$\text{Vertical steel required at water face} = \frac{M}{\sigma_{st}\, j\, d}$$

$$= \frac{18.0 \times 10^6}{150 \times 0.874 \times 235}$$

$$= 585\ \text{mm}^2$$

Provide ϕ 12 @ 180 mm c/c ($A_t = 628\ \text{mm}^2$) vertical bars on inner face of wall.

Vertical steel required away from the water face

$$= \frac{11.7 \times 10^6}{190 \times 0.892 \times 235}$$

$$= 294\ \text{m}^2$$

Provide ϕ 12 @ 300 mm c/c ($A_t = 377\ \text{mm}^2$) vertical bars on outer face of wall.

Provide ϕ 10 @ 250 mm c/c horizontal bars on both faces of wall.

(3) *Design of Roof Slab*

Assuming the thickness of slab as 150 mm.

Self weight of slab (150 mm) = $0.15 \times 25 = 3.75\ \text{kN/m}^2$

Live load $= 1.50\ \text{kN/m}^2$

Total load, $w = 5.25\ \text{kN/m}^2$

c/c dimensions of slab = $(3.275 \times 6.675\ \text{m})$

As the aspect ratio of slab is more than 2, it behaves as a one-way slab.

$$\text{Therefore, maximum BM in slab} = \frac{5.25 \times 3.275^2}{8} = 7.0\ \text{kN}$$

$$\text{Effective depth required, } d = \sqrt{\frac{7.0 \times 10^6}{1.012 \times 1000}}$$

$$= 84\ \text{mm} < 110\ \text{mm} \quad \text{(O.K.)}$$

$$\text{Area of steel required, } A_t = \frac{7.0 \times 10^6}{190 \times 0.892 \times 110}$$

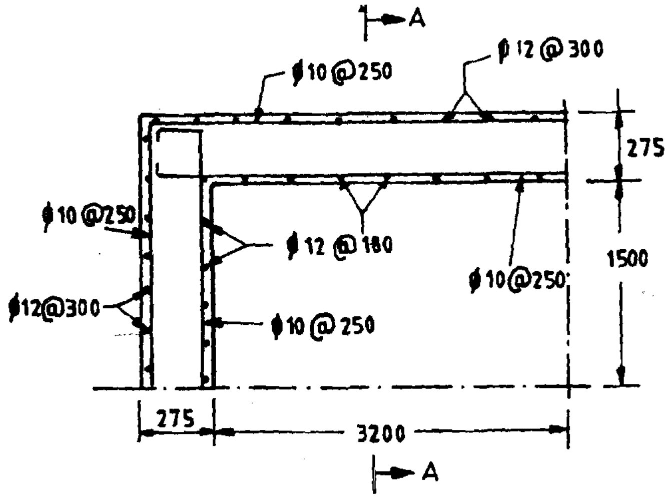
$$= 376\ \text{mm}^2$$

Provide ϕ 10 @ 200 mm c/c along short span and ϕ 10 @ 250 mm c/c along long span.

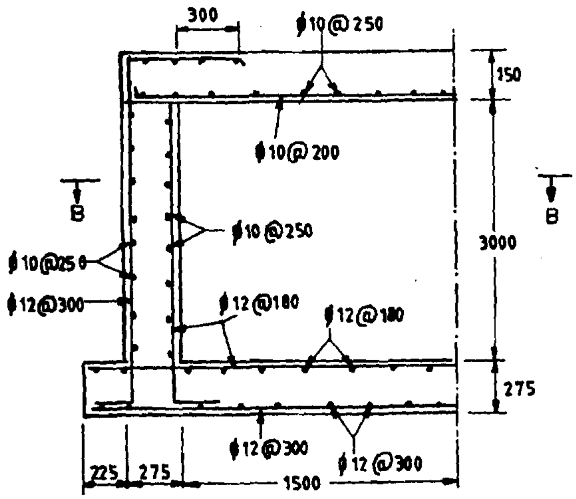
(4) *Design of Base Slab*

The BM at the edges of the slab will be same as the BM at the base of the wall. Therefore, providing 275 mm thick base slab with ϕ 12 @ 180 mm c/c (bothways) at the top and ϕ 12 @ 300 mm c/c (bothways) at the bottom.

The reinforcement detail is shown in Figure 22.18.



(a) Sectional Plan (Sec. B–B)



(b) Sectional Elevation (Sec. A–A)

Figure 22.18 : Reinforcement Details

22.5 SUMMARY

A reinforced concrete water tank can be situated on the ground, under ground or above ground level. The tank may be either open or roofed over and may be either circular or rectangular in plan.

Besides strength, water tightness is one of the main considerations in the design of reinforced water tanks. This can be achieved by proper mixing, placing and curing of concrete having well graded aggregate, water cement ratio less than 0.5 and richer proportions of cement. In no case, cracking of concrete on the water face is desirable. Minimum grade of cencrete used in water tanks is M 20.

22.6 ANSWERS TO SAQs

Refer the relevant preceding text in the unit or other useful books on the topic listed in the section "Further Reading" to get the answers of the SAQs.

FURTHER READING

IS : 456 - 1986, "*Indian Standard Code of Practice for Plain and Reinforced Concrete*", (Third Revision).

O. P. Jain and Jaikrishna, "*Plain and Reinforced Concrete Structures*", Vol. I & II, Nemchand Bros., Roorkee.

A. K. Jain, "*Limit State Design of Reinforced Concrete*", Nemchand Bros., Roorkee.

N. Krishna Raju, "*Design of Reinforced Concrete Structures*", CBS Publishers, Delhi.

N. Krishna Raju, "*Design of Bridges*", Oxford & IHB Publication, New Delhi.

V. K. Raina, "*Concrete Bridge Practice : Analysis, Design & Economy*", Tata McGraw Hill, New Delhi.

S. K. Mallik and A. P. Gupta, "*Reinforced Concrete*", Oxford & IBH Publication, Calcutta.