# UNIT 10 WATER TANKS

## Structure



## 10.1 INTRODUCTION

A water tank is used for storage and distribution of water over its command area (Figures 10.1 to 10.3). Tanks may be of rectangular, cylindrical, polygonal or of any other shape depending upon their capacities, economy and architectural requirements. Tanks may be :

(a) Underground water tanks,

(b) Surface tanks resting on ground, and

(c) Overhead water tanks.

**Figure 10.1 : Overground Tank** **Figure 10.2 : Underground Tank**

The support for an overhead tank may be a group of columns with intermediate bracings. The bracings are provided for resisting horizontal forces as well as for reducing effective length of columns. For larger capacity tanks, staging may be in the form of single shaft *circular or polygonal* in plan and may be tapering. The foundation may be isolated footing, or annular footing with or without beam, or raft (Figure 10.3). For simplicity of analysis and design, only those types of tanks and staging have been described which may be analysed by membrane theory.

**(a)** **(b)**

**Figure 10.3 : Overhead Tanks with different Types of Staging**

## Objectives

After studying this unit, you should be able to

- describe the water tanks and its components,
- classify the water tanks, and
- explain the design steps of the cylindrical tanks resting on ground with flexible base, underground cylindrical tanks with flexible base, and overhead intze type tank with circular shaft staging.

# 10.2 DESIGN AND DETAILING SPECIFICATIONS OF WATER TANKS

## 10.2.1 Specifications

Water tanks must be leak proof. The designed sections of its components must be crack resistant, and even if the cracks are formed they must be very fine and evenly distributed, so that the full section may be effective to prevent leak and to protect reinforcement from corroding.

To meet above requirements IS : 3370 (Part I to IV), Code of Practice for Concrete Structures for Storage of Liquids, prescribes the following:

(a) The elements shall be designed by Working Stress Method assuming full cross section (including cover) uncracked and effective by allowing limited tensile strength of concrete $\delta_{ct}$ (Table 10.1).

**Table 10.1 : Permissible Concrete Stress in Calculation Relating to Resistance to Cracking**

| Grade of Concrete ($N/mm^2$) | Permissible Stresses Direct Tension |
|---|---|
| M 20 | 1.2 |
| M 25 | 1.3 |
| M 30 | 1.5 |
| M 35 | 1.6 |
| M 40 | 1.7 |

(b) Reduced tensile strength of reinforcement (Table 10.2) are taken to indirectly offset the effect of corrosion.

**Table 10.2 : Permissible Stresses in Steel Reinforcement for Strength Calculation**

| Sl. No. | Type of Stress in Steel Reinforcement | Permissible Stresses in $N/mm^2$ | |
|---|---|---|---|
| | | Plain Round Mild Steel Bars Conforming to Grade I of IS : 432 (Part I ) – 1966 | High Yield Strength Deformed Bars Conforming to IS : 1786-1966 IS : 1139-1966 |
| (1) | (2) | (3) | (4) |
| 1. | Tensile stress in members under direct tension | 115 | 150 |

(c) Increased nominal covers than those as per IS : 456 – 2000 are provided so that the surface cracks may not reach up to reinforcement.

*For water faces* of parts of members either in contact with the water or enclosing the space above the water (such as inner faces of roof slab), the minimum *cover* to all reinforcement should be 25 mm or the diameter of the main bar, whichever is greater.

In the presence of sea water, soils and water of corrosive character the *cover* should be increased by 12 mm but this additional cover shall not be taken into account for design calculations.

*For faces away from the water* and for parts of the structure neither in contact with the water on any face nor enclosing the space above the water, the cover shall conform to the requirements of IS : 456-2000 (Refer Unit 1).

*In case of mild steel* the minimum reinforcement in walls, floors and roofs in each of two directions at right angles shall have an area of 0.3 percent of the concrete section in that direction for sections upto 100 mm thick. For sections of thickness greater than 100 mm and less than 450 mm the minimum reinforcement in each of the two directions shall be linearly reduced from 0.3 percent for 100 mm thick section to 0.2 percent for 450 mm thick section. For sections of thickness greater than 450 mm, minimum reinforcement in each of the two directions shall be kept at 0.2 percent. In concrete section of thickness 225 mm or greater, two layers of reinforcing steel shall be placed one near each face of the section to make up the minimum reinforcement specified in this clause.

*In case of high yield strength deformed bars* the minimum reinforcement specified above may be decreased by 20 percent.

In no case the percentage of reinforcement in any member shall be less than that specified in IS : 456-2000.

Other design and detailing requirements such as compressive stresses, bar size, development length etc. shall comply with the provisions of IS : 456 - 2000 (Refer Unit 1).

## 10.2.2 Design Procedure for Overground Cylindrical Tanks with Flexible Base

**Size of Tank**

$$\text{Volume, } V = \frac{\pi}{4} D^2 H$$

where $D$ = Inside diameter of tank, and

$H$ = Total depth of tank.

Since $D$ and $H$ are both unknowns, therefore, either of the two is assumed to get the other. Here $H$ = Depth of water + Free board.

**Design of Wall**

*Hoop Reinforcement* ($A_s$)

$$\text{Maximum Hoop Tension, } T = w H \frac{D}{2}$$

where $w$ = Density of water (kN/m$^3$), and

$T$ = Hoop tension/m height.

Equating Applied Force, $T$ = Resisting force (tensile steel $A_s$ only taken)

$$T = \sigma_{st} A_s$$

$$\text{or, Hoop Reinforcement/m, } A_s = \frac{T}{\sigma_{st}}$$

**Thickness of Wall (*t*)**

Equating Applied Force, $T$ = Resisting force (whole cross section including $A_s$ is taken).

or, $$T = \delta_{ct} \{b\, t + (m - 1) A_s\}$$

where $\delta_{ct}$ = Permissible direct tensile stress in concrete (Table 10.1),

$b$ = breadth of section in vertical direction (usually taken 1 m),

$t$ = thickness of section, and

$m$ = modular ratio.

Thickness of wall is usually not less than 150 mm.

**Minimum Reinforcement ($A_{s,\min}$)**

The hoop reinforcement as well as temperature and shrinkage reinforcement in the vertical direction shall not be less than $A_{st,\min}$ (10.2.1).

## Curtailment of Hoop Reinforcement

Theoretically hoop reinforcement requirement gradually reduces and approaches to zero at the top. Hence curtailment becomes necessary specially where the tank is deep. This is explained through Example 10.1.

## Design of Base Slab

The pressure on the ground due to water, wt. of wall and self weight of base slab is almost uniformly distributed over the whole base area and it is equal and opposite to the uniformly distributed reaction. Hence, a minimum thickness of slab with minimum reinforcement is provided for base slab. To make the joint between wall and base slab flexible, such that the wall may deform at the junction without any restraint, the wall and the base are separated from each other and the joint is filled with bitumen (Figures 10.1 and 10.2).

A screed or concrete layer of 75 mm thick of concrete grade M 10* shall be first laid on the ground and covered with a sliding layer of bitumen paper or other suitable material to destroy the bond between the screed and base slab.

* In case of aggressive soil or injurious subsoil water, M 15 concrete with sulphate resisting cement shall be provided as the screed.

### Example 10.1

Design a circular water tank with dome as top cover resting over ground for a capacity of 200,000 litres. Depth of the tank is to be 3.2 m including 0.2 m free board. Use M 30 concrete and Fe 250 steel.

## Solution

### Size of Tank

Depth of water in the tank = 3.2 – 0.2 = 3 m

If $D$ = inside diameter of tank, then $\dfrac{200,000 \times 10^3}{10^6} = \dfrac{\pi}{4} D^2 \times 3$

1 litre = 1000 cc $= \dfrac{10^3}{10^6}\ m^3$

or, $D = 9.213$ m

Hence, provided $D = 9.25$ m.

### Design of Wall

*Hoop Reinforcement* ($A_s$)

Maximum hoop tension, $T = wH\dfrac{D}{2}$

or, $$T = 10 \times 3.2 \times \frac{9.25}{2} = 148 \text{ kN}$$

$$\sigma_{st} = 115 \text{ N/mm}^2 \text{ from Table 10.2}$$

$A_s$ required at the *base* level

$$A_s = \frac{T}{\sigma_{st}} = \frac{148 \times 1000}{115} = 1287 \text{ mm}^2/\text{m}$$

Hence provided $\phi 12$ @ 85 mm c/c = 1329 mm$^2$/m.

*Thickness of Wall* ($t$)

From equilibrium of forces,

$$T = \delta_{ct}\{bt + (m-1)A_s\}$$

or, $148 \times 10^3 = 1.5 \times \{1000 \times t + (9 - 1) \times 1329\}$

or, $t = 88.035$

Provided thickness of wall, $t$ = 150 mm.

*Temperature and Shrinkage reinforcement*

$$p_t\% = 0.3 - \frac{(0.3 - 0.2)}{(450 - 100)} \times (150 - 100) = 0.286\ \%$$

$$\therefore \quad A_{s,\min} = \frac{0.286}{100} \times 1000 \times 150 = 429\ \text{mm}^2/\text{m}$$

Provided $\phi$ 10 @ 180 mm c/c.

These vertical bars will also act as tie bars for main reinforcement.

**Curtailment of Hoop Reinforcement**

Half of the hoop reinforcement may be curtailed at half the depth of the tank, i.e. at 1.6 m depth as the variation of $A_s$ is proportional to $H$. The curtailed bars, however will be extended by greater of $\phi$ 12 and $d$ (Figure 10.4).

**Design of Base Slab**

Provided total depth of slab = 150 mm and temperature and shrinkage reinforcement same as that for wall of 150 mm thickness, i.e. $\phi$ 10 @ 180 mm c/c.

The reinforcement detailing is shown in Figure 10.4.

**Figure 10.4 : Reinforcement Detailing**

## 10.2.3 Design Procedure for Underground Cylindrical Tanks with Flexible Base

The cylindrical wall, in this case, will be subjected to

(a) Water pressure only from inside when the tank is full and earth is not filled from outside,

(b) Earth pressure only from outside when the tank is empty, and

(c) Water pressure from inside and earth pressure from outside simultaneously in normal circumstances.

The above two cases, (a) and (b) are generally critical and design of wall will be guided by these two.

Other design and detailing specifications will remain the same discussed in Section 10.2.2.

**Example 10.2**

Design the same water tank as in Example 10.1 but lying underground. Angle of repose = 25° and soil density = 17 kN/m$^3$.

**Solution**

**Size of Tank**

Total height, $H$ = 3.2 m, and inside diameter, $D$ = 9.25 m.

**Design of Wall**

(a) when water pressure is *only* considered, the design will be same as done in Example 10.1.

(b) when earth pressure *only* is acting.

**Hoop Reinforcement**

Maximum hoop compression = $k_a \gamma H \dfrac{D}{2}$

$$= \frac{1 - \sin 25}{1 + \sin 25} \times 17 \times 3.2 \times \frac{9.25}{2} = 102.114 \text{ kN}$$

Compressive Stress, $\sigma_{cc} = \dfrac{102.114 \times 10^3}{1000 \times t}$

or, $8 = \dfrac{102.114 \times 10^3}{1000 \times t}$ (for M30 concrete, $\sigma_{cc}$ = 8 N/mm$^2$)

or $t$ = 12.895 mm << 150 taken in Example 10.1.

Hence, the design and detailing will remain same as for Example 10.1 (Figure 10.4).

## 10.2.4 Design Procedure for Overhead Intze Type Tanks

**Size of Tank**

Most economical and structurally efficient tank size is determined by the formula :

$$V = 0.585\, D_o^3$$

where $V$ = Total volume of the Intze tank (Figure 10.5), and

$D_o$ = Inside diameter of the tank.

All other dimensions are given in terms of $D_o$

As this type of tank is provided only when the tank size is comparatively large. Flat slab for base as well as for top cover become costlier propositions with respect to those for domes including the cost for shuttering.

**Figure 10.5 : Fixing Size**

The shape of the bottom dome is conical so that the horizontal components of force at the junction at the base will be of opposite sign to that for the spherical dome base to make the base ring beam economical.

## Design of Top Dome

The section of dome is very thin in comparison to its diameter. From membrane analysis, the hoop as well as meridional stresses are compressive only up to $51.8^{o}$ semi-central angle (Figure 10.6(a)).

**Figure 10.6(a) : Semi-central Angle**

The rise of the dome is taken as 1/5 th to 1/8 th of the diameter of the tank. The thickness is taken between 60 mm and 150 mm depending upon the diameter of the tank. Once the diameter, rise and thickness of the dome are fixed, the other dimensional data such as semi central angle (θ) and the radius of the sphere (*R*) and the load data may be calculated.

**Figure 10.6(b) : Meridional Thrust and Hoop Force**

For Design (Figure 10.6(b)).

Meridional thrust, $N_\phi = \dfrac{wR}{1 + \cos\theta}$

and Corresponding compressive stress $= \dfrac{N_\phi}{bt}$

where $w$ = load per unit area on dome, and

$b$ = width of section (generally taken as 1 m).

The meridional compressive stress is checked against permissible compressive stress. Generally permissible stress is greater than the actual one, hence only nominal reinforcement is provided along meridian.

Next, Hoop thrust, $N_\phi = w\,R \left(\cos\theta - \dfrac{1}{1+\cos\theta}\right)$

$\therefore$ Hoop stress $= \dfrac{N_\theta}{bt}$

Generally, hoop stress is less than the permissible compressive stress and, therefore, only nominal hoop reinforcement is provided.

### Design of Top Ring Beam

It is designed for the horizontal component of maridonial force at the bottom of the dome acting radially. The hoop force, $T$, developed due to above radial force is given by the formula (Figure10.6(c)).

$$T = (N_\phi \cos\theta)\, r_1$$

where $r$ = Radius of the ring beam.

**Figure 10.6(c) : Components of Meridional Force**

The rest of the design procedure is the same as that for a tank wall.

### Design of Vertical Wall

It is the same as that for vertical wall in Section 10.2.2.

### Design of Ring Beam at the Junction of Vertical and Conical Dome (Figure 10.7)

It is designed for the horizontal component of the axial force in conical dome,

$$P = W \cot\theta$$

where $W$ = Total Vertical load above the beam including its selfweight.

The rest of the design procedure is the same as that for wall of a cylindrical wall.

**Figure 10.7 : Force at the Junction of Vertical Wall and Conical Dome**

### Design of Conical Dome

An element of the conical dome will be designed for meridional force (i.e. axial force) as well as for lateral force perpendicular to the surface of the dome (Figure 10.8).

**Figure 10.8 : Forces for Design of Conical Dome**

### Design of Bottom Dome

It is similar to top dome except that the load is due to weight of water above it including its self weight.

### Design of Bottom Ring Beam

It is similar to the top ring beam except that the design radial pressure is the difference of the horizontal component of forces for bottom spherical and conical domes.

The whole design procedure has been explained through Example 10.3.

## SAQ 1

(a) Draw free hand sketches of elevated water tanks with

  (i) column type staging, and

  (ii) with circular shaft type staging.

(b) Why water tanks are designed by working stress method?

(c) Write short notes on specifications which are applicable to water tanks only.

(d) Why the base of an underground or overground tank made flexible? How it is done? Explain with a sketch.

(e) Why an Intze type tank becomes economical? How economically efficient dimensions are fixed?

(f) How a dome is designed?

## 10.3 DESIGN SPECIFICATIONS FOR CIRCULAR SHAFT TYPE STAGING

**Thickness of Shaft (*t*)**

Thickness of shaft shall not be less than 150 mm. If the internal diameter of the shaft is more than 6 m,

$$t \geq 150 + \frac{D - 6000}{120}$$

where $D$ = Internal diameter of the shaft in mm.

**Reinforcement in Shaft**

*Vertical Reinforcement*

The diameter of bars shall not be less than 10 mm. A minimum of 0.25% reinforcement in two layers shall be provided. The spacing of bars shall neither be more 2 $t$ or 400 whichever is less.

*Circumferential Reinforcement*

A minimum of 0.2% reinforcement in two layers shall be provided. In no case the reinforcement shall be less than 400 $mm^2$/m height. The spacing of bars shall neither be more than $t$ nor more than 300, whichever is less. *These reinforcement shall be nearer to the faces*.

**Cover**

Inside nominal cover shall not be less than 25, however that for outside not less 40.

**Analysis of Shaft**

For analysis, the loads acting on the shaft are evaluated as explained below :

(a) *Vertical loads* include load of the supported tank, weight of water of its full capacity, self weight of shaft and imposed load on top dome of the tank.

(b) *Horizontal loads* may be either due to wind or due to earthquake*.

* Earthquake load has not been included here for simplicity.

Wind pressure is evaluated as per IS : 875-1987 Part III. The total horizontal load is equal to wind pressure multiplied by projected area of tank including staging. However, this load is modified by a factor called 'Shape Factor' whose value depends upon the shape of the tank and its staging.

For determining the *critical* stresses due to vertical and horizontal loads following combinations shall be considered :

(a) *All vertical (gravity) loads*,

(b) *All vertical loads excluding imposed load with tank empty + wind load, and*

(b) *All vertical loads excluding* imposed load with tank full + wind load.

Whole cross section of shaft shall be in compression if

$$e \leq \frac{r}{2}$$

where $e$ = The eccentricity of the load = $\frac{M}{W}$

$$= \frac{\text{Moment in vertical plane at the section under consideration}}{\text{Total vertical load above section under consideration}}$$

In such case, the maximum vertical compressive stress in concrete shall be given by

$$\sigma_{cv} = \frac{W}{A} + \frac{M}{I}\, r$$

or, $$\sigma_{cv} = \frac{W}{2\pi r t}\left(1 + \frac{2e}{r}\right)$$

where $A = 2\,\pi\, r\, t$

$I_x = I_r = \pi\, r^3\, t$

and $\sigma_{cv}$ = Maximum vertical stress in concrete at outside diameter of shaft shell in N/mm$^2$.

**Permissible Stresses ***

*Refer IS : 11682-1985 : criteria for design of RCC staging for overhead tanks.

*Concrete*

The stress in concrete shall not exceed the following limits for various combination of loads :

[**Note :** If shell thickness is adequate to satisfy Cl. (e) of permissible stresses of concrete this requirement may be waived.]

| **Combination of Loads** | **Stress Limit** |
|---|---|
| (a) Dead load + wind load | 0.38 $\sigma_{cv}$ |
| (b) Dead load + earthquake forces | 0.40 $\sigma_{cv}$ |
| (c) Circumferential tensile stress in concrete due to wind induced ring moment | 0.07 $\sigma_{cv}$ |

Here $\sigma_{cv}$ = 28-day ultimate cube strength of concrete in N/mm$^2$.

*Reinforcement*

The stresses in steel shall not exceed the following limits for various combination of loads :

| **Combination of Loads** | **Stress Limit** |
|---|---|
| (a) Dead load + Wind load | 0.57 $\sigma_{sy}$ |
| (b) Dead load + Earthquake Loads | 0.60 $\sigma_{sy}$ |

| | |
|---|---|
| (c) Circumferential tensile steel due to wind induced ring moment | 0.50 $\sigma_{sy}$ |

Here $\sigma_{sy}$ = yield or proof stress of steel in N/mm$^2$.

The reinforcement detailing is shown in Figure 10.9.

**Figure 10.9 : Explaining Specifications for Reinforcement Detailing**

**SAQ 2**

Write short notes on specifications for the design of a circular shaft type staging.

## 10.4 DESIGN OF ANNULAR FOOTING

Vertical or gravity load is the total load on footing including its self weight. The moment due to horizontal load about ground level is also considered for the design.

The maximum pressure ($f_{BC}$) on the soil is given by formula

$$f_{BC} = \frac{W}{A} + \frac{M}{I} \cdot \frac{D_e}{2}$$

where $D_e$ = External diameter of annular footing for safety and stability, and

$f_{BC} \leq \sigma_{Bc}$ (i.e. bearing capacity of soil).

**Example 10.3**

An overhead Intze type water tank is to be designed for a capacity of 600 kl for a staging height above GL of 15 m. Design parameters are as follows :

Bearing capacity of soil = 200 kN/m$^2$; $f_{ck}$ = 25 N/mm$^2$ for tank portion and 20 N/mm$^2$ for staging $f_y$ = 250 N/mm$^2$ for tank and $f_y$ = 415 N/mm$^2$ for staging.

## Solution

### Fixing Size

Total capacity = 600kl

$$600 = 0.585\, D_o^{\,3}$$

or, $D_o = 10.085$ m

**Provided $D_o = 10$ m**

### Inside Surface Dimension of the Tank

Refer Figures 10.5 and 10.10.

**Figure 10.10 : Dimension of the Tank**

Radius of cylindrical portion $= \dfrac{D_0}{2} = r_1 = 5$ m

Rise of Top dome $h_1 = 2$ m

Rise of Bottom dome $h_2 = 1$ m

Chord of Bottom dome = $\dfrac{5}{8} D_0 = 2\, r_2 = 6$ m

Depth of cylindrical tank = $\dfrac{2}{3} D_0 = h = 7$ m

Height of conical dome = $\dfrac{3}{16} D_0 = h_3 = 2$ m

### Capacity Calculation

Total volume of water = Volume of cylindrical portion + Volume of conical portion – Volume of the bottom dome portion

$$= V = Q_1 + Q_2 - Q_3$$

$$= \pi\, r_1^{\,2}\,(h - 0.15) + \pi\,(r_1^{\,2} + r_1\, r_2 + r_2^{\,2})\,\frac{h_3}{3} - \pi\,(3r_2^{\,2} + h_2^{\,2})\,\frac{h_2}{6}$$

where free board = 0.15 m.

$$= \pi\,[(5^2 \times (7 - 0.15) + (5^2 + 5 \times 3 + 3^2)\frac{2}{3} - (3 \times 3^2 + 1^2) \times \frac{1}{6})]$$

$$= 625.962 \text{ m}^3 > 600 \text{ m}^3$$

**Hence, dimension chosen are OK.**

**Design of Top Dome**

Let thickness of dome = 125 mm

Radius of dome = $R_t = \dfrac{r_1^2 + h_1^2}{2h_1} = \dfrac{(5^2 + 2^2)}{2 \times 2} = 7.25$ m.

The surface area of the dome

$$S_t = 2\,\pi\,R_t\,h_1 = 2\,\pi \times 7.25 \times 2 = 91.106 \text{ m}^2$$

$$\sin\theta = \frac{5}{7.25} = 0.6897$$

or $\theta = \sin^{-1} 0.6897 = 43.61$

$$\cos\theta = \cos 43.61 = 0.724$$

**Load Calculation**

Self wt., $w_s = 0.125 \times 1 \times 1 \times 25 = 3.125$ kN/m$^2$

$IL$, $w_i = 0.75$ kN/m$^2$

Total load on the dome/m$^2$ $= 3.875$ kN/m$^2$

Total $IL = W_1 = 2\,\pi\,R_t\,h_1\,w_i = 2\pi \times 7.25 \times 2 \times 0.75 = 68.34$ kN

Total $DL = W_2 = 2\pi\,R_t\,h_1 w_s = 2\pi \times 7.25 \times 2 \times 3.125 = 284.7$ kN

Total ($IL + DL$) on top dome $= 353.04$ kN

**Meridional Thrust**

$$N_\phi = \frac{w R_t}{1 + \cos\theta} = \frac{3.875 \times 7.25}{(1 + 0.724)} = 16.296 \text{ kN/m}$$

$\therefore$ Compressive stress $= \dfrac{N_\phi}{1 \times t} = \dfrac{16.296 \times 10^3}{10^3 \times 125} = 0.13$ N/mm$^2$ << 6 N/mm$^2$

Hence, OK.

**Hoop Thrust**

$$N_\theta = w\,R_t \left(\cos\theta - \frac{1}{1 + \cos\theta}\right)$$

$$= 3.875 \times 7.25 \times \left(0.724 - \frac{1}{(1 + 0.724)}\right)$$

$$= 4.044 \text{ kN/m}$$

Hoop stress $= \dfrac{4.044 \times 10^3}{10^3 \times 125} = 0.03$ N/mm$^2$ << 6 N/mm$^2$

Hence, OK.

*Nominal Reinforcement*

For 125 thickness $p\% = 0.3 - \dfrac{(0.3-0.2)}{(450-100)} \times (125-100)$

$= 0.293\%$

$\dfrac{100\ A_{st}}{1000 \times 125} = 0.293$

or, $A_{st} = 366.25\ \text{mm}^2/\text{m}$

Hence, provided $\phi$ 10 @ 210 mm c/c bothways.

Design of top ring beam (Figure 10.11).

**Figure 10.11 : Component of Meridional Force at Base for Ring Beam Design**

$$T = (N_\phi \cos\theta)\ r_1 = 16.296 \times 0.724 \times 5 = 58.99\ \text{kN}$$

$$A_s = \frac{T}{\sigma_{st}} = \frac{58.99 \times 10^3}{115} = 513\ \text{mm}^2$$

Provided 8 $\phi$10 ($A_s = 8 \times 78 = 624\ \text{mm}^2$)

Assuming $b = 250$

for M 25, $m = \dfrac{280}{3\sigma_{cbc}} = \dfrac{280}{3 \times 8.5} = 10.98 \approx 11$

$$T = \sigma_{ct}\ (b\,D + (m-1)\,A_{st})$$

$$58.99 \times 10^3 = 1.3\ (250 \times D + (11-1) \times 624)$$

or, $D = 156.55$ mm

**Provided $D$ = 250 mm and shear stirrups of $\phi$ 6 @ 300 mm c/c.**

Wt. of ring beam $= 2\pi\left(5 + \dfrac{0.250}{2}\right) \times 0.250 \times 0.25 \times 25$

$= 50.315$ kN.

### Design of Vertical Wall

Height of wall = 7 m

Hoop tension (maximum) = $(wh)\ r_1 = 10 \times 7 \times 5 = 350$ kN

Hoop Tension at 6 m = $(wh)\ r_1 = 10 \times 6 \times 5 = 300$ kN

$\therefore$ Average force for 1 m depth, i.e. from 6.0 m to 7.0 m $= \dfrac{(350+300)}{2}$

$= 325$ kN.

$$A_s = \frac{325 \times 10^3}{115} = 2826\ \text{mm}^2$$

Provided $\phi$ 16 @ 140 mm c/c on each face, i.e. $A_s = 2871\ \text{mm}^2$

$$T = \sigma_{ct}\,(bD + (m - 1)\,A_{st})$$

$$325 \times 10^3 = 1.3\,(1000\,D + (11 - 1) \times 2871)$$

or, $D = 221.29$ mm

Provided $D = 225$ mm

**Nominal Reinforcement in Vertical Direction**

For 225 thickness $p\% = 0.3 - \frac{(0.3 - 0.2)}{(450 - 100)} \times (225 - 100)$

$= 0.264\%$

or, $\frac{100\,A_{st}}{1000 \times 225} = 0.264$

or, $A_{st} = 594$ mm$^2$/m.

**Hence, provided $\phi$ 10 @ 270 mm c/c on both faces.**

**Design of Middle Ring Beam**

Taking cross sectional dimension = 500 × 500

wt. of top dome = 353.040 kN

wt. of top ring beam = 50.315 kN

wt. of cylindrical portion = 2π (5.113 × 0.225 × 7 × 25) = 1265 kN

wt of ring beam at the junction of cylindrical and conical dome

= 2π × 5.25 × 0.5 × 0.5 × 25 = 206.16 kN

∴ Total load on top of conical dome, $w = 1874.5$ kN

∴ Load/m run $= \frac{1874.5}{2\pi\,(5 + 0.113)} = 58.34$ kN/m.

$T \cos \theta = P$ and $T \sin \theta = W$

or, $P = W \cot \theta = 58.34 \times 1 = 58.34$ kN (Taking $\theta = 45^\circ$)

Hoop tension in ring beam = 58.34 × 5.113 = 298.29 kN

∴ $A_s = \frac{298.29 \times 10^3}{115} = 2593.82$ mm$^2$

**Hence, provided 16 $\phi$ 16.**

Hoop tension = $298.29 \times 10^3 = \sigma_{ct}\,(bD + (m - 1)\,A_s)$

or $D = \left\{ \frac{298.29 \times 10^3}{1.3} - (11 - 1) \times 3216 \right\} \div 500$

$= 394.58$

**Hence, Provided cross-sectional area of Middle Ring Beam = 500 × 500.**

**Design of Conical Dome**

Taking thickness of conical dome = 250 mm

Depth at mid height = 7 + 1 = 8 m

Mean radius of ring at mid height = 5 – 1 = 4 m

Thickness of conical dome = 250 mm

Total vertical load upto the middle ring beam = 1874.5 kN weight of conical portion up to the ring of 1 m length at mid height of the conical dome

$$= 2\pi \times \frac{(5+4)}{2} \times 0.25 \times 2\sqrt{2} \times 25 = \underline{499.74 \text{ kN}}$$

Sub-total = 2374.24 kN

Weight of water of rectangular cross-section + that of $\Delta^{ar}$ cross-section

$$= 10 \times \pi \times 4.5 \times 7 \times 1 + \frac{10}{2} \times \pi \times 4.5 \times 1 \times 1 = \underline{1060.287}$$

kN

Total = 34340.53 kN

Load/m $= \frac{3434.53}{\pi \times 4.5} = 242.94$ kN/m

$\therefore \quad T_2 = 242.94 \text{ cosec } \theta = 242.94 \times \sqrt{2} = 343.52$ kN/m.

Horizontal component of the thrust

$$H_1 = T_2 \cos\theta = 343.52 \times \frac{1}{\sqrt{2}} = 242.94 \text{ kN/m}$$

$\therefore \quad$ Meridional stress $= \frac{343.52 \times 10^3}{1000 \times 250} = 1.37 \text{ N/mm}^2 < 6 \text{ N/mm}^2$

Vertical pr. on the slab per unit horizontal area ($m^2$)

= weight of water + weight of slab component

$= 10 \times 8 + 0.25 \times 1 \times \sqrt{2} \times 25 = 88.83 \text{ kN/m}^2$

on horizontal area.

Normal pressure on unit length of the conical length

$$= \frac{1}{\sqrt{2}} \times \frac{1}{\sqrt{2}} \times 88.83 = 44.41 \text{ kN/m}^2$$

Hoop Tension $= \frac{pD}{2} = 44.41 \times 4 = 177.66$ kN

$$A_s = \frac{177.66 \times 10^3}{115} = 1544.86 \text{ mm}^2\text{/m}$$

**Hence, Provided $\phi$ 12 @ 140 mm c/c ($A_s = 1614 \text{ mm}^2$) in two layers.**

Hoop Force $= 177.66 \times 10^3 = \sigma_{ct} \{bD + (m-1) A_s\}$

or, $\quad D = \left\{\frac{177.66 \times 10^3}{1.3} - (11-1) \times 1614\right\} \div 1000$

$= 120.52 < 250.$

**Hence, provided thickness of conical dome slab = 250 mm.**

**Nominal Reinforcement in Axial direction**

For 250 thickness $p\% = 0.3 - \frac{(0.3-0.2)}{(450-100)} \times (250-100)$

$= 0.26\%$

$$\frac{100\,A_s}{bd} = 0.26$$

or $$A_s = \frac{0.26 \times 1000 \times 250}{100} = 650$$

**Hence, provided $\phi$ 10 @ 240 mm c/o in each face.**

**Design of Bottom Dome**

Half chord length, i.e. $r_2 = 3$ m

Rise of dome $h_2 = 1$ m

Thickness of dome = 150 mm

Radius of dome $R_b = \frac{1}{2h_2}(r_2^2 + h_2^2) = \frac{1}{2 \times 1}(3^2 + 1^2) = 5$ m

Self wt. = $(2\pi\, R_b\, h_2)\, t_2\, \gamma_c = 2\pi \times 5 \times 1 \times 0.15 \times 25$ = 117.81 kN

Wt. of water on the dome = $\gamma \pi \left[ r_2^2 h' - (3r_2^2 + h_2^2)\frac{h_2}{6} \right]$

$$= 10\,\pi \left[ 3^2 \times 9 - (3 \times 3^2 + 1^2)\frac{1}{6} \right] = 2398.08 \text{ kN}$$

Total wt. on dome, $w$ = 2515.89 kN

Semicentral angle $\theta = \sin^{-1}\frac{3}{5} = 36.87^{\circ}$

$$w = \frac{W}{\text{Surface area}} = \frac{W}{2\pi R_b h_2} = \frac{2515.89}{2\pi \times 5 \times 1} = 80.08 \text{ kN/m}^2$$

Meridional thrust, $N_\phi = \frac{wR_b}{(1+\cos\theta)} = \frac{80.08 \times 5}{(1+\cos 36.87)} = 222.44 \text{ kN/m}^2$

Meridional stress $= \frac{N_\phi}{bt} = \frac{222.44 \times 10^3}{1000 \times 150} = 1.48 \text{ N/m}^2 < 6 \text{ N/mm}^2$

Hoop compression = $wR \left( \cos\theta - \frac{1}{1+\cos\theta} \right)$

$$= 80.08 \times 5 \times \left( \cos 36.87 - \frac{1}{1+\cos 36.87} \right) = 97.87 \text{ kN/m}^2$$

Hoop stress = $\frac{97.87 \times 10^3}{1000 \times 150} = 0.652 \text{ N/mm}^2 < 6 \text{ N/mm}^2$

**Nominal Reinforcement**

For 150 thickness $p\% = 0.3 - \frac{(0.3-0.2)}{(450-100)} \times (150-100) = 0.286\%$

$$\frac{100\,A_s}{bd} = 0.286$$

or, $$A_s = \frac{0.286 \times 1000 \times 150}{100} = 429$$

**Hence, provided ϕ 10 @ 180 mm c/c both ways.**

Reinforcement detailing for the tank has been shown in Figure 10.12.

### Design of Circular Shaft Staging

Diameter of shaft = 6 m

Height of shaft above GL = 15 m

Thickness of shaft wall above GL = 150 mm

Depth of shaft below GL = 2 m

Thickness of shaft below GL = 400 mm

**Figure 10.12 : Reinforcement Detailing of Tank**

### Load acting on Shaft at GL

| | |
|---|---|
| Load of top dome including imposed load | = 353.04 kN |
| Load of top ring beam | = 50.315 kN |
| Load of tank | = 1265 kN |
| Weight of middle ring beam | = 206.12 kN |
| Load of conical dome | = 499.74 kN |
| Load of bottom spherical dome | = 117.81 kN |
| Load of bottom Ring Beam | |
| $= 2\pi \times 3 \times 0.5 \times 0.5 \times 25$ | = 117.80 kN |

Self wt. of shaft from bottom ring beam

to GL = π × 6 × 15 × 0.15 × 25 = 1060.29 kN

Sub-total = 3670.11 kN

Self wt. of shaft from GL to top of footing

π × 6 × 2 × 0.4 × 25 = 376.99 kN

Sub-total = 4047.10 kN

wt. of water from capacity calculation = 6259.62 kN

Total = 10306.72 kN

## Wind Load

Shape Factor = 0.7

| Sl. No. | Segment | Wind Pr kN/m$^2$ | Area m$^2$ | Distance of CG from GL(m) | Moment About GL (kN) | Distance of CG about Foundation Level | Moment about Foundation Level |
|---|---|---|---|---|---|---|---|
| 1. | Top dome | 1.0 | 13.7 | 24.834 | 238.16 | 26.834 | 257.34 |
| 2. | Cylindrica l Portion of tank | 1.0 | 70 | 20.5 | 1004.5 | 22.5 | 1102.5 |
| 3. | Trapezium | 1.0 | 16 | 16.083 | 180.13 | 18.083 | 202.53 |
| 4. | Supporting Cyl.shaft upto GL | 1.0 | 90 | 7.5 | 472.5 | 9.5 | 598.5 |
| | | | | | $\sum MGL$ =1895.29 kN | | $\sum M_{FDN}$ = 2160.87 |

*Eccentricity, e, of the Load When Tank is Empty*

$$e = \frac{\sum M_{GL}}{W_{GL}} = \frac{1895.29}{3670.11} = 0.506 \text{ m} < \left(\frac{6}{4} = 1.5 \text{ m}\right)$$

Hence there will be compressive stress on the whole cross section in all cases as eccentricity ($e$) has been calculated for the lightest load.

*Stress at GL When Tank is Empty*

$$\sigma_{cv} \text{ at GL when tank empty} = \frac{W}{2\pi rt}\left(1 + \frac{2e}{r}\right)$$

$$= \frac{3670.11}{2\pi \times 3 \times .150}\left(1 + \frac{2 \times 0.516}{3}\right) \times \frac{10^3}{10^6}$$

$= 1.74 \text{ N/mm}^2 < (0.38 \times 25 = 9.5 \text{ N/mm}^2)$

*Eccentricity e of the Load When Tank is Full*

$$e = \frac{M_{GL}}{W_{CL}} = \frac{1895.29}{(3670.11 + 6259.62)} = 0.19$$

$$\sigma_{cv} \text{ at when tank full} = \frac{(3670.11 + 6259.62)}{2\pi \times 3 \times 0.15}\left(1 + \frac{2 \times 0.19}{3}\right) \times \frac{10^3}{10^6}$$

$$= 3.96 < 6 \text{ N/mm}^2$$

*When Tank Empty at Foundation Level*

$$e = \frac{2160.87}{4047.1} = 0.53 \text{ m} < \left(\frac{6}{4} = 1.5 \text{ m}\right)$$

$\sigma_{cv}$ at FDN Level when tank empty

$$= \frac{40471}{2\pi \times 3 \times 0.4}\left(1 + \frac{2 \times 0.53}{3}\right) \times \frac{10^3}{10^6}$$

$$= 0.726 < (0.38 \times 25 = 9.5 \text{ N/mm}^2)$$

When tank full at foundation Level

$$e = \frac{2160.87}{10306.72} = 0.209$$

$\sigma_{cv}$ at foundation level when tank is full

$$= \frac{10306.72}{2\pi \times 3 \times 0.4}\left(1 + \frac{2 \times 0.209}{3}\right)\frac{10^3}{10^6} = 1.56 < 6 \text{ N/mm}^2$$

The reinforcement detailing will be done as per Section 10.3.

**Design of Foundation (Figure 10.13)**

Total load on footing = 10306.72 kN

Self wt 10% = 1030.67 kN

Total wt = 11337.39 kN

$$f_{Bc} = \frac{W}{A} + \frac{M_x}{I_x} \cdot \frac{D}{2} = \frac{11337.39}{\frac{\pi}{4}(10^2 - 1^2)} + \frac{2160.87}{\frac{\pi}{64}(10^4 - 1^4)} \times 5$$

$$= 145.81 + 22.013$$

$$= 167.82 \text{ kN/m}^2 < 200 \text{ kN/m}^2.$$

**Design of Cantilever Slab**

Taking 1m strip

Effective Span = 2 m

**Figure 10.13 : Plan of Foundation**

## Estimate of Total Depth

*From Deflection Control*

$$\frac{l_{ef}}{d} \le k_B\, k_1\, k_2\, k_3\, k_4$$

where $k_B = 7$, $k_1 = 1$

$k_2$ for M 20 and Fe 415 = 1

$$k_3 = k_4 = 1$$

$$d \ge \frac{l_{ef}}{k_B\, k_1\, k_2\, k_3\, k_4} = \frac{2 \times 10^3}{7 \times 1 \times 1 \times 1 \times 1} = 285.714$$

Taking $D = 600$ mm; $d = 600 - 50 - \frac{20}{2} = 540$ mm

*From Moment of Resistance Consideration*

**Loads**

Loads from reaction = 167.82 – 0.6 × 1 × 1 × 25

= 152.82 kN/m$^2$

Design load = $w_u$ = 1.5 × 152.82 = 229.23 kN/m$^2$

$$\text{Design Moment} = M_u = \frac{W_u {l_{ef}}^2}{2} = \frac{229.23 \times 2^2}{2}$$

= 458.46 kN m/m

$$M_{u,\,lim} = 458.46 \times 10^6$$

$$= 0.36 \times 0.48\,(1 - 0.42 \times 0.48)\, 1000 \times d^2 \times 20$$

$$d = 407.61 \text{ mm} < 540 \text{ mm}$$

Hence, OK.

***$A_{st}$***

$$M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{bdf_{ck}}\right)$$

$$458.46 \times 10^6 = 0.87 \times 415 \times A_{st} \times 540\left(1 - \frac{A_{st} \times 415}{1000 \times 540 \times 20}\right)$$

$$458.46 \times 10^6 = 194967\, A_{st} - 7.492\, {A_{st}}^2$$

or, ${A_{st}}^2 - 26023.35\, A_{st} + 61193272.8 = 0$

or, $A_{st} = \dfrac{26023.35 \pm \sqrt{(26023.35^2 - 4 \times 6119372.8}}{2}$

$= 2614.06 \text{ mm}^2$

Provided 20 ϕ @ 120 mm c/c $\left( \dfrac{1000 \times 314}{120} = 2617 \right)$

Check for shear at *d* from face of support

$$V_u = w_n \, l_{ef} = 229.23 \times (2 - 0.54)$$

$$= 334.67 \text{ kN}$$

$$\tau_v = \frac{V_u}{bd} = \frac{334.67 \times 10^3}{1000 \times 540} = 0.62 \text{ N/mm}^2$$

$$\frac{100\, A_{st}}{bd} = \frac{100 \times 2617}{1000 \times 540} = 0.484\%$$

Corresponding M 20 and $\tau_c = 0.62$

$$\frac{100\, A_{st}}{bd} = 1\%$$

or, $A_{st} = \dfrac{1000 \times 540}{100} = 5400 \text{ mm}^2/\text{m}$

**Hence, provided ϕ 20 @ 55 mm (Figure 10.14).**

**Figure 10.14 : Reinforcement Detailing of Foundation**

[**Note :** Same design will be applicable for inside portion of footing as the pressure will reduce substantially.]

## 10.5 SUMMARY

The design and detailing of simple types of water tanks – circular water tanks with flexible base (both under and over ground) and Intze type overhead water tank with cylindrical staging and annular foundation – have been described.

*Only* direct stresses (i.e. compression or tension) are developed in their components due to applied forces and, hence, principles and specifications of analysis and design are very simple. To make the tanks leak proof, corrosion resistant and durable, working stress method of design with higher grades of concrete, reduced permissible stresses in steel, larger amount of temperature and shrinkage reinforcement and thicker cover to reinforcement have been prescribed by the Codes.

## 10.6 ANSWERS TO SAQs

**SAQ 1**

(a) Refer Section 10.1.

(b) Refer Section 10.2.

(c) Refer Section 10.2.

(d) Refer Section 10.2.

(e) Refer Section 10.2.4.

(f) Refer Section 10.2.4.

**SAQ 2**

Refer Section 10.3.

# FURTHER READING

BIS : 456-2000, *Code of Practice for Plain and Reinforced Concrete*, Bureau of Indian Standards, Manak Bhawan, Bahadur Shah Zafar Marg, New Delhi.

Ashok K. Jain, *Reinforced Concrete Limit State Design*, New Chand and Brothers, Roorkee.

S. K. Mallick and A. P. Gupta, *Reinforced Concrete*, Oxford and IBH Publishing Company Private Limited.

S. N. Sinha, *Reinforced Concrete Design*, Tata McGraw-Hill Publishing Company Limited, Asaf Ali Road, Delhi.

S. U. Pillai and D. Menon, *Reinforced Concrete Design*, Tata McGraw-Hill Publishing Company Limited, West Petal Nagar, New Delhi.

# THEORY OF STRUCTURES-II

Construction has been an activity which has witnessed many civilizations. Different construction materials and techniques have been tried in the past. In modern times, for construction of any type of structure generally the choice for material is confined to either concrete or steel. Concrete, though strong in compression, is extremely weak in tension. Steel, on the other hand, is very strong in tension as well as in compression. Therefore, the combination of concrete and steel has proved to be most suitable choice to withstand stresses.

The widespread use of reinforced concrete in a variety of structural members in different type of structures has compelled a proper understanding of the design and detailing procedures. All reinforced concrete structures need proper designing taking into account tensile and compressive stresses, shears, creep and thermal effect, etc. This course, entitled "Theory of Structures-II", covers the key aspects of design and detailing of different reinforced concrete structures.

This course comprises ten units.

In Unit 1, you will be introduced to the limit state method of design of reinforced concrete structures or their elements and limit state of flexural collapse.

Unit 2 deals with the method of design of beams and slabs for shear and torsion. It also discusses concrete reinforcement and detailing.

In Unit 3, the principles of design and detailing have been applied for the design and detailing of simply supported rectangular beam, cantilever beam and simply supported flanged beams.

In Unit 4, the design and detailing of cantilever slab, one-way simply supported slabs, two-way simply supported slabs and two-way restrained slabs are described.

Unit 5 introduces you to the design and detailing of axially loaded rectangular and circular columns.

In Unit 6, design and detailing of strip footings, isolated footings and combined footings are discussed.

Unit 7 deals with the planning as well as structured design and detailing of flights with or without stringer beams of rectangular staircases.

Unit 8 discusses the design and detailing of reinforced concrete cantilever type of retailing wall.

In Unit 9, working stress method of design for structures and their elements is explained.

Finally, the design and detailing of circular water tank with flexible base (both under and overground) and Intze type overhead water tank with cylindrical staging and annular foundation are described in Unit 10.

A number of Self-Assessment Questions (SAQs) are given in each unit to help you to self monitor your own progress. You are advised to study the text carefully. Try to solve the SAQs on your own and verify your answers with those given at the end of each unit. This will definitely develop your confidence.

At the end, we wish you all the best for your all educational endeavours.