CHAPTER 8

WATER TANKS

In general there are three kinds of water tanks-tanks resting on ground, underground tanks and elevated tanks.

The tanks resting on ground like clear water reservoirs, settling tanks, aeration tanks etc. are supported on the ground directly. The walls of these tanks are subjected to pressure and the base is subjected to weight of water and pressure of soil. The tanks may be covered on top.

The tanks like purification tanks, Imhoff tanks, septic tanks, and gas holders are built underground. The walls of these tanks are subjected to water pressure from inside and the earth pressure from outside. The base is subjected to weight of water and soil pressure. These tanks may be covered at the top.

Elevated tanks are supported on staging which may consist of masonry walls, R.C.C. tower or R.C.C. columns braced together. The walls are subjected to water pressure. The base has to carry the load of water and tank load. The staging has to carry load of water and tank. The staging is also designed for wind forces.

From design point of view the tanks may be classified as per their shape-rectangular tanks, circular tanks, intze type tanks. spherical tanks conical bottom tanks and suspended bottom tanks.

Design requirement of concrete (I.S.I)

In water retaining structures a dense impermeable concrete is required therefore, proportion of fine and course aggregates to cement should be such as to give high quality concrete.

Concrete mix weaker than M200 is not used. The minimum quantity of cement in the concrete mix shall be not less than 300 kg/m^3.

The design of the concrete mix shall be such that the resultant concrete is sufficiently impervious. Efficient compaction preferably by vibration is essential. The permeability of the thoroughly compacted concrete is dependent on water cement ratio. Increase in water cement ratio increases permeability, while concrete with low water cement ratio is difficult to compact. Other causes of leakage in concrete are defects such as segregation and honey combing. All joints should be made water-tight as these are potential sources of leakage.

Design of liquid retaining structures is different from ordinary R.C.C, structures as it requires that concrete should not crack and hence tensile stresses in concrete should be within permissible limits.

A reinforced concrete member of liquid retaining structures is designed on the usual principles ignoring tensile resistance of concrete in bending. Additionally it should be ensured that tensile stress on the liquid retaining face of the equivalent concrete section does not exceed the permissible tensile strength of concrete as given in table

1. For calculation purposes the cover is also taken into concrete area.

Cracking may be caused due to restraint to shrinkage, expansion and contraction of concrete due to temperature or shrinkage and swelling due to moisture effects. Such restraint may be caused by –

(i) the interaction between reinforcement and concrete during shrinkage due to drying.

(ii) the boundary conditions.

(iii) the differential conditions prevailing through the large thickness of massive concrete.

Use of small size bars placed properly, leads to closer cracks but of smaller width. The risk of cracking due to temperature and shrinkage effects may be minimised by limiting the changes in moisture content and temperature to which the structure as a whole is subjected. The risk of cracking can also be minimised by reducing the restraint on the free expansion of the structure with long walls or slab founded at or below ground level, restraint can be minimised by the provision of a sliding layer. This can be provided by founding the structure on a flat layer of concrete with interposition of some material to break the bond and facilitate movement.

In case length of structure is large it should be subdivided into suitable lengths separated by movement joints, specially where sections are changed the movement joints should be provided.

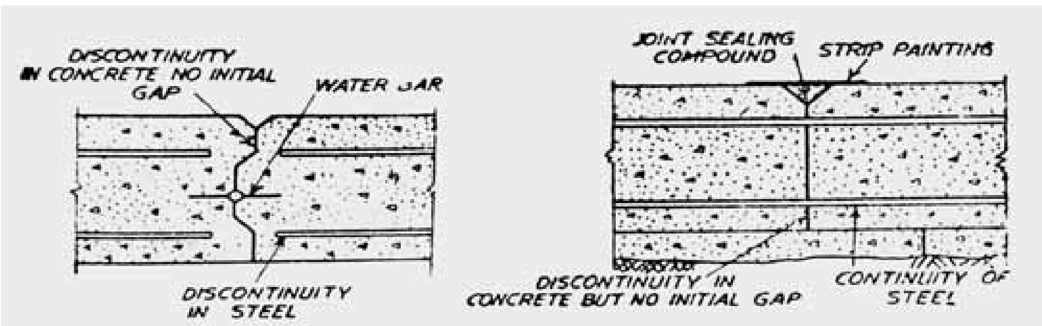
Where structures have to store hot liquids, stresses caused by difference in temperature between inside and outside of the reservoir should be taken into account.

The coefficient of expansion due to temperature change may be taken as 11 x 10^{-6}/° C and coefficient of shrinkage may be taken as 450 x 10^{-6} for initial shrinkage and 200 x 10^{-6} for drying shrinkage.

3. Joints in Liquid Retaining Structures. Joints are classified as given below.

(a) Movement Joints. There are three types of movement joints.

(i) ***Contraction Joint*.** It is a movement joint with deliberate discontinuity without initial gap between the concrete on either side of the joint. The purpose of this joint is to accommodate contraction of the concrete. The joint is shown in Fig. 1(a).



(a) (b)

Fig 1.

A contraction joint may be either complete contraction joint or partial contraction joint. A complete contraction joint is one in which both steel and concrete are interrupted and a partial contraction joint is one in which only the concrete is interrupted, the reinforcing steel running through as shown in Fig. 1(b).

(ii) ***Expansion Joint.*** It is a joint with complete discontinuity in both reinforcing steel and concrete and it is to accommodate either expansion or contraction of the structure. A typical expansion joint is shown in Fig. 2.

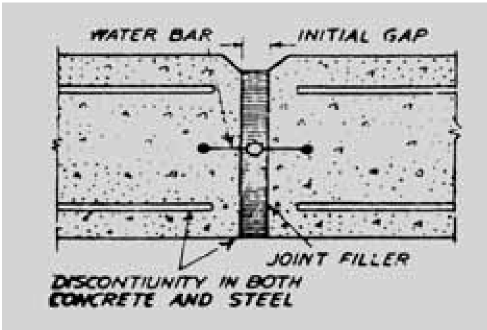
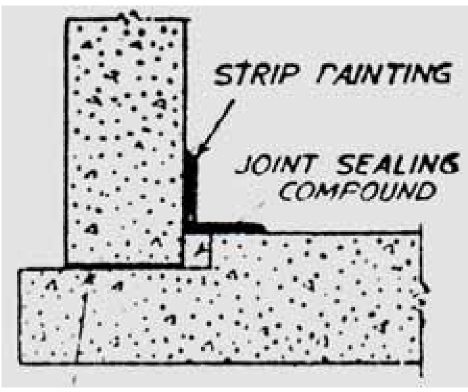


Fig. 2

This type of joint requires the provision of an initial gap between the adjoining parts of a structure which by closing or opening accommodates the expansion or contraction of the structure.

(iii) ***Sliding Joint***. It is a joint with complete discontinuity in both reinforcement and concrete and with special provision to facilitate movement in plane of the joint. A typical joint is shown in Fig. 3. This type of joint is provided between wall and floor in some cylindrical tank designs.

(b) ***Construction Joint***. This type of joint is provided for convenience in construction. Arrangement is made to achieve subsequent continuity without relative movement. One application of these joints is between successive lifts in a reservoir wall. A typical joint is shown in Fig. 4.



RCPARED SILDING SURFACE
OR RUBBER PAO

Fig. 3

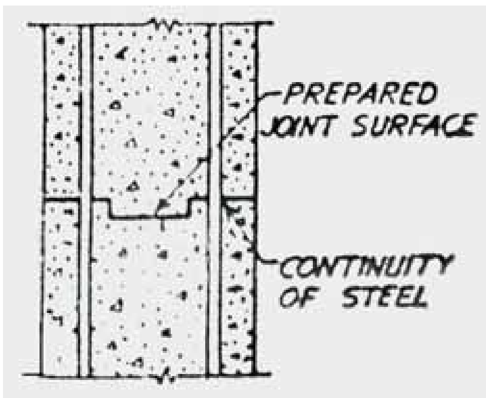


Fig. 4

The number of joints should be as small as possible and these joints should be kept from possibility of percolation of water.

(c) ***Temporary Open Joints***. A gap is sometimes left temporarily between the concrete of adjoining parts of a structure which

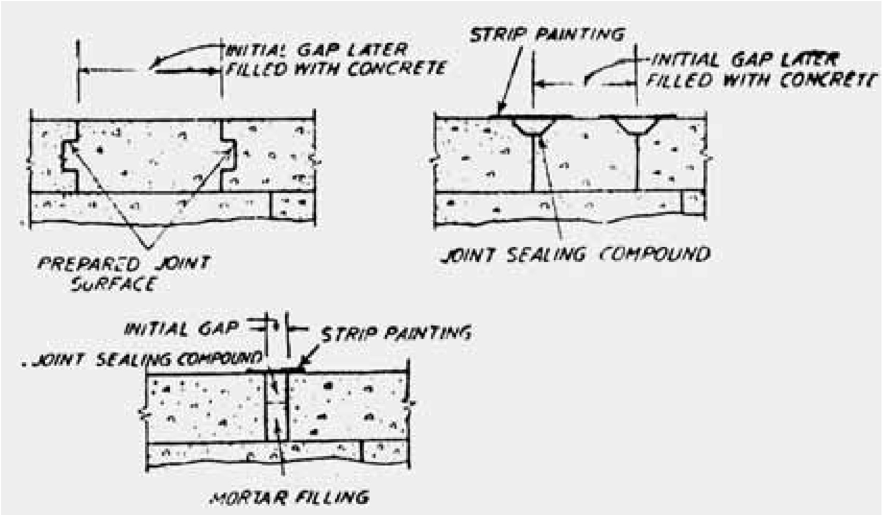


Fig. 5 Temporary open joints

after a suitable interval and before the structure is put to use, is filled with mortar or concrete completely as in Fig. 5(a) or as shown in Fig. 5 (b) and (c) with suitable jointing materials. In the first case width of the gap should be sufficient to allow the sides to be prepared before filling.

Spacing of Joints. Unless alternative effective means are taken to avoid cracks by allowing for the additional stresses that may be induced by temperature or shrinkage changes or by unequal settlement, movement joints should be provided at the following spacings:-

(a) In reinforced concrete floors, movement joints should be spaced at not more than 7.5 m apart in two directions at right angles. The wall and floor joints should be in line except where sliding joints occur at the base of the wall in which correspondence is not so important.

(b) For floors with only nominal percentage of reinforcement (smaller than the minimum specified) the concrete floor should be cast in panels with sides not more than 4.5 m.

(c) In concrete walls, the movement joints should normally be placed at a maximum spacing of 7.5 m. in reinforced walls and 6m. in unreinforced walls. The maximum length desirable between vertical movement joints will depend upon the tensile strength of the walls, and may be increased by suitable reinforcement. When a sliding layer is placed at the foundation of a wall, the length of the wall that can be kept free of cracks depends on the capacity of wall section to resist the friction induced at the plane of sliding. Approximately the wall has to stand the effect of a force at the place of sliding equal to weight of half the length of wall multiplied by the co-efficient of friction.

(d) Amongst the movement joints in floors and walls as mentioned above expansion joints should normally be provided at a spacing of not more than 30 m. between successive expansion joints or between the end of the structure and the next expansion joint; all other joints being of the construction type.

(e) When, however, the temperature changes to be accommodated are abnormal or occur more frequently than usual as in the case of storage of warm liquids or in uninsulated roof slabs, a smaller spacing than 30 m should be adopted, that is greater proportion of movement joints should be of the expansion type). When the range of temperature is small, for example, in certain covered structures, or where restraint is small, for example, in certain elevated structures none of the movement joints provided in small structures up to 45m. length need be of the expansion type. Where sliding joints are provided between the walls and either the floor or roof, the provision of movement joints in each element can be considered independently.

4. General Design for Requirements (I.S.I)

1. Plain Concrete Structures. Plain concrete member of reinforced concrete liquid retaining structures may be designed against structural failure by allowing tension in plain concrete as per the permissible limits for tension in bending. This will automatically take care of failure due to cracking. However, nominal reinforcement shall be provided, for plain concrete structural members.

2. Permissible Stresses in Concrete.

(a) For resistance to cracking. For calculations relating to the resistance of members to cracking, the permissible stresses in tension (direct and due to bending) and shear shall confirm to the values specified in Table 1. The permissible tensile stresses due to bending apply to the face of the member in contact with the liquid. In members less than 225 mm. thick and in contact with liquid on one side these permissible stresses in bending apply also to the face remote from the liquid.

(b) For strength calculations. In strength calculations the permissible concrete stresses shall be in accordance with Table 1. Where the calculated shear stress in concrete alone exceeds the permissible value, reinforcement acting in conjunction with diagonal compression in the concrete shall be provided to take the whole of the shear.

Table 1
Permissible concrete stresses in calculations relating to resistance to cracking

Grade of concrete	Permissible stress in kg/cm^2 Tension		Shear (= Q/bjd)
	Direct	Due to Bending	
M 150	11	15	15
M 200	12	17	17
M 250	13	18	19
M 300	15	20	22
M 350	16	22	25
M 400	17	24	27

3. Permissible Stresses in Steel

(a) For resistance to cracking. When steel and concrete are assumed to act together for checking the tensile stress in concrete for avoidance of crack, the tensile stress in steel will be limited by the requirement that the permissible tensile stress in the concrete is not exceeded so the tensile stress in steel shall be equal to the product of modular ratio of steel and concrete, and the corresponding allowable tensile stress in concrete.

(b) For strength calculations. In strength calculations the permissible stress shall be as follows:

(i) Tensile stress in member in direct tension — 1000 kg/cm^2

(ii) Tensile stress in member in bending on liquid retaining face of members or face away from liquid for members less than 225 mm thick. — 1000 kg/cm^2

On face away from liquid for members 225 mm. or more in thickness. — 1250 kg/cm^2

(iii) Tensile stress in shear reinforcement, For members less than 225 mm thickness — 1000 kg/cm^2

For members 225 mm or more in thickness — 1250 kg/m^2

(iv) Compressive stress in columns subjected to direct load. — 1250 kg/cm^2

Note 1. Stress limitations for liquid retaining faces shall also apply to:

(a) Other faces within 225 mm of the liquid retaining face.

(b) Outside or external faces of structures away from the liquid but placed in water logged soils upto the level of highest subsoil water level.

Note 2. The permissible stress of 1000 kg/cm^2 in (i), (ii) and (iii) may be increased to 1125 kg/cm^2 in case of deformed bars and in case of plain mild steel bars when the cross reinforcement is spot welded to the main reinforcement.

4. Stresses due to drying Shrinkage or Temperature Change.

(i) Stresses due to drying shrinkage or temperature change may be ignored provided that –

(a) the permissible stresses specified above in (ii) and (iii) are not otherwise exceeded.

(b) adequate precautions are taken to avoid cracking of concrete during the construction period and until the reservoir is put into use.

(c) recommendation regarding joints given in article 8.3 and for suitable sliding layer beneath the reservoir are complied with, or the reservoir is to be used only for the storage of water or aqueous liquids at or near ambient temperature and the circumstances are such that the concrete will never dry out.

(ii) Shrinkage stresses may however be required to be calculated in special cases, when a shrinkage co-efficient of 300 x 10^{-6} may be assumed.

(iii) When the shrinkage stresses are allowed, the permissible stresses, tensile stresses to concrete (direct and bending) as given in Table 1. may be increased by 33 per cent.

5. Floors

(i) Provision of movement joints. Movement joints should be provided as discussed in article 3.

(ii) Floors of tanks resting on ground. If the tank is resting directly over ground, floor may be constructed of concrete with norminal percentage of reinforcement provided that it is certain that the ground will carry the load without appreciable subsidence in any part and that the concrete floor is cast in panels with sides not more than 4.5 m. with contraction or expansion joints between. In such cases a screed or concrete layer less than 75 mm thick shall first be placed on the ground and covered with a sliding layer of bitumen paper or other suitable material to destroy the bond between the screed and floor concrete.

In normal circumstances the screed layer shall be of grade not weaker than M 100, where injurious soils or aggressive water are expected, the screed layer shall be of grade not weaker than M 150 and if necessary a sulphate resisting or other special cement should be used.

(iii) Floor of tanks resting on supports

(a) If the tank is supported on walls or other similar supports the floor slab shall be designed as floor in buildings for bending moments due to water load and self weight.

(b) When the floor is rigidly connected to the walls (as is generally the case) the bending moments at the junction between the walls and floors shall be taken into account in the design of floor together with any direct forces transferred to the floor from the walls or from the floor to the wall due to suspension of the floor from the wall.

If the walls are non-monolithic with the floor slab, such as in cases, where movement joints have been provided between the floor slabs and walls, the floor shall be designed (only for the vertical loads on the floor.

(c) In continuous T-beams and L-beams with ribs on the side remote from the liquid, the tension in concrete on the liquid side at the face of the supports shall not exceed the permissible stresses for controlling cracks in concrete. The width of the slab shall be determined in usual manner for calculation of the resistance to cracking of T-beam, L-beam sections at supports.

(d) The floor slab may be suitably tied to the walls by rods properly embedded in both the slab and the walls. In such cases no separate beam (curved or straight) is necessary under the wall, provided the wall of the tank itself is designed to act as a beam over the supports under it.

(e) Sometimes it may be economical to provide the floors of circular tanks, in the shape of dome. In such cases the dome shall be designed for the vertical loads of the liquid over it and the ratio of its rise to its diameter shall be so adjusted that the stresses in the dome are, as far as possible, wholly compressive. The dome shall be supported at its bottom on the ring beam which shall be designed for resultant circumferential tension in addition to vertical loads.

6. Walls

(i) Provision of Joints

(a) Sliding joints at the base of the wall. Where it is desired to allow the walls to expand or contract separately from the floor, or to prevent moments at the base of the wall owing to fixity to the floor, sliding joints may be employed.

(b) The spacing of vertical movement joints should be as discussed in article 8.3 while the majority of these joints may be of the partial or complete contraction type, sufficient joints of the expansion type should be provided to satisfy the requirements given in article

(ii) Pressure on Walls.

(a) In liquid retaining structures with fixed or floating covers the gas pressure developed above liquid surface shall be added to the liquid pressure.

(b) When the wall of liquid retaining structure is built in ground, or has earth embanked against it, the effect of earth pressure shall be taken into account.

(iii) Walls or Tanks Rectangular or Polygonal in Plan.

While designing the walls of rectangular or polygonal concrete tanks, the following points should be borne in mind.

(a) In plane walls, the liquid pressure is resisted by both vertical and horizontal bending moments. An estimate should be made of the proportion of the pressure resisted by bending moments in the vertical and horizontal planes. The direct horizontal tension caused by the direct pull due to water pressure on the end walls, should be added to that resulting from horizontal bending moments. On liquid retaining faces, the tensile stresses due to the combination of direct horizontal tension and bending action shall satisfy the following condition:

$$\frac{t'}{t} + \frac{\sigma'_{ct}}{\sigma_{ct}} \leq 1$$

where,

t' = calculated direct tensile stress in concrete.

t = permissible direct tensile stress in concrete (Table 1)

σ'_{ct} = calculated tensile stress due to bending in concrete.

σ_{ct} = permissible tensile stress due to bending in concrete.

(d) At the vertical edges where the walls of a reservoir are rigidly joined, horizontal reinforcement and haunch bars should be provided to resist the horizontal bending moments even if the walls are designed to withstand the whole load as vertical beams or cantilever without lateral supports.

(c) In the case of rectangular or polygonal tanks, the side walls act as two-way slabs, whereby the wall is continued or restrained in the horizontal direction, fixed or hinged at the bottom and hinged or free at the top. The walls thus act as thin plates subjected triangular loading and with boundary conditions varying between full restraint and free edge. The analysis of moment and forces may be made on the basis of any recognized method.

(ii) **Walls of Cylindrical Tanks**. While designing walls of cylindrical tanks the following points should be borne in mind:

(a) Walls of cylindrical tanks are either cast monolithically with the base or are set in grooves and key ways (movement joints). In either case deformation of wall under influence of liquid pressure is restricted at and above the base. Consequently, only part of the triangular hydrostatic load will be carried by ring tension and part of the load at bottom will be supported by cantilever action.

(b) It is difficult to restrict rotation or settlement of the base slab and it is advisable to provide vertical reinforcement as if the walls were fully fixed at the base, in addition to the reinforcement required to resist horizontal ring tension for hinged at base, conditions of walls, unless the appropriate amount of fixity at the base is established by analysis with due consideration to the dimensions of the base slab the type of joint between the wall and slab, and, where applicable, the type of soil supporting the base slab.

7. Roofs

(i) Provision of Movement Joints. To avoid the possibility of sympathetic cracking it is important to ensure that movement joints in the roof correspond with those in the walls, if roof and walls are monolithic. It, however, provision is made by means of a sliding joint for movement between the roof and the wall correspondence of joints is not so important.

(ii) Loading. Field covers of liquid retaining structures should be designed for gravity loads, such as the weight of roof slab, earth cover if any, live loads and mechanical equipment. They should also be designed for upward load if the liquid retaining structure is subjected to internal gas pressure.

A superficial load sufficient to ensure safety with the unequal intensity of loading which occurs during the placing of the earth cover should be allowed for in designing roofs. The engineer should specify a loading under these temporary conditions which should not be exceeded. In designing the roof, allowance should be made for the temporary condition of some spans loaded and other spans unloaded, even though in the final state the load may be small and evenly distributed.

(iii) Water tightness. In case of tanks intended for the storage of water for domestic purpose, the roof must be made water-tight. This may be achieved by limiting the stresses as for the rest of the tank, or by the use of the covering of the waterproof membrane or by providing slopes to ensure adequate drainage.

(iv) Protection against corrosion. Protection measure shall be provided to the underside of the roof to prevent it from corrosion due to condensation.

8. Minimum Reinforcement

(a) The minimum reinforcement in walls, floors and roofs in each of two directions at right angles shall have an area of 0.3 per cent of the concrete section in that direction for sections upto 100 mm, thickness. 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 sections. For sections of thickness greater than 450 mm, minimum reinforcement in each of the two directions shall be kept at 0.2 per cent. In concrete sections of thickness 225 mm or greater, two layers of reinforcement steel shall be placed one near each face of the section to make up the minimum reinforcement.

(b) In special circumstances floor slabs may be constructed with percentage of reinforcement less than specified above. In no case the percentage of reinforcement in any member be less than 0°15% of gross sectional area of the member.

9. Minimum Cover to Reinforcement.

(a) For liquid faces of parts of members either in contact with the liquid (such as inner faces or roof slab) the minimum cover to all reinforcement should be 25 mm or the diameter of the main bar whichever is grater. In the presence of the sea water and soils and water of corrosive characters the cover should be increased by 12 mm but this additional cover shall not be taken into account for design calculations.

(b) For faces away from liquid and for parts of the structure neither in contact with the liquid on any face, nor enclosing the space above the liquid, the cover shall be as for ordinary concrete member.

5. Tanks Resting on Ground. For small capacities rectangular tanks are generally used and for bigger capacities circular tanks are used. The walls of circular tanks may have flexible joints or rigid joints at the base.

6. Ciruclar Tanks with Flexible Joint at the Base. In these tanks walls are subjected to hydrostatic pressure. The tank wall is designed as thin cylinder.

At the base, minimum pressure

$= wH$.

This causes hoop tension

$$= \frac{wHD}{2}$$

where

w is the density of water.

H is depth of water, and

D is diameter of the tank

Steel area required at the base for one metre height

$$= \frac{wHD}{2 \times 1000} cm^2$$

If '*t*' is the thickness of wall, tensile stress in concrete

$$= \frac{wHD/2}{100t + (m-1)A_t} kg/cm^2$$

As the hoop tension reduces gradually to zero at top, the reinforcement is gradually reduced to minimum reinforcement at top. The main reinforcement consists of circular hoops Vertical reinforcement equal to 0.3% of concrete area is provided and hoop reinforcement is tied to this reinforcement. In smaller tanks main reinforcement is placed near the outer face. For bigger tanks the wall thickness is more and the reinforcement is placed on both faces.

Though assumed that base is flexible, but in reality there will always be some restraint at the base and some pressure will be resisted by cantilever action of the wall. The minimum vertical reinforcement provided will be adequate to resist bending stresses caused by cantilever action.

Example 1. Design a circular tank with flexible base for capacity of 500,000 litres.

Sol. Depth of 4 m, is provided with free board of 20 cm. If *D* is the diameter or the tank capacity of the tank will be

$$\frac{\pi}{4} \times D^2 \times 3.8 = \frac{500{,}000 \times 10^3}{10^6}$$

$$D = \sqrt{\frac{500 \times 4}{3.8\pi}} = 12.94m.$$

Provide diameter of 13 m.

Maximum hoop tension at base for one metre height

$$= \frac{wHD}{2} = \frac{1000 \times 4 \times 13}{2} = 26{,}000kg$$

Area of steel required

$$= \frac{26{,}000}{1000} = 26cm^2$$

Provide 16 mm ϕ bars at 7 cm centres.

$$A_t = 28.73 \text{ cm}^2$$

Use mix *M* 200. Allowable stress in tension

$$= 12 \text{ kg/cm}^2.$$

Let '*t*' be thickness of wall.

Tensile stress = $\dfrac{26{,}000}{100t + (13-1)\times 28.73} = 12$

$$t = \frac{26{,}000}{12\times 100} - \frac{12\times 28.73}{100}$$

$= 21.67 - 3.448 = 25.118$ cm

Provide thickness of 26 cm.

Hoop reinforcement will be provided on both faces. 16 mm ϕ bars at 14 cm centres are provided on each face.

Vertical reinforcement

$$= \left[0.3 - 0.1\times\frac{(26-10)}{35}\right]\%$$

$= 0.26\%$

Bending moment

$$M = \frac{EId^2y}{dx^2} =, \text{shear force F} = EI\frac{d^3y}{dx^3}$$

Loading intensity

$$P_c = EI\frac{d^4y}{dx^4}$$

If the effect of the lateral restraint is taken into account modified flexural rigidity will be

$$= \frac{Et^3}{12(1-\mu^2)} \qquad (\mu = \text{Poisson's ratio})$$

$$\therefore \frac{Et^3}{12(1-\mu^2)}\frac{d^4y}{dx^4} = w(H-x) - \frac{4tEy}{D^z}$$

$$\therefore \frac{d^4y}{dx^4} + \frac{48(1-u^2)y}{t^2D^2} = \frac{12w(1-u^2)(H-x)}{Et^3}$$

Putting $\alpha = 4\sqrt{\dfrac{1(1-u^2)}{t^2D^2}}$

$$\frac{d^4y}{dx^4} - 4x^4y = \frac{12w(H-x)(1-\mu^2)}{Et^3}$$

The solution of this differential equation is

$$y - e^{\alpha z}(C_1\cos\alpha x + C_2\,\text{Sin}\,\alpha x) + e^{-\alpha z}(C_3\cos\alpha x + C_4\sin\alpha x) + \frac{w(H-x)D^2}{4Et}$$

The values of constants C_1, C_2, C_3 and C_4 will depend on the restraint provided at top and bottom.

The value of μ may be taken as 0.2.

1. Circular Tanks Fixed at Base and Free at top. At top the shear force and *B.M* will be zero. At the base slope and deflection will be zero. Applying these four conditions, four equations will be obtained which can be solved and constants C_1, C_2, C_3 and C_4 evaluated.

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

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

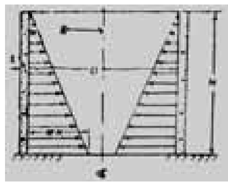
$$At\ x = 0, \quad \theta = 0 \quad \therefore EI\frac{dy}{dx} = 0$$

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

Knowing four constant solution of elastic curve is known. Hence values of P_c and P_r can be found at different heights. Table gives coefficients for ring tension and *B.M* at various heights and shear at base.

Table 2.A
CYLNDRICAL TANKS WITH FIXED BASE, FREE TOP

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



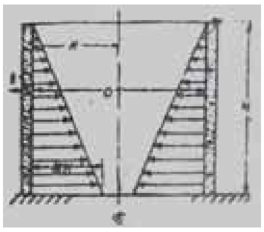
H^2	Coefficient at Point									
D_t	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H
0.4	+0.149	+0.134	+0.120	+0.101	+0.062	+0.066	+0.049	+0.029	+0.014	+0.004
0.2	+0.263	+0.239	+0.215	+0.190	+0.160	+0.130	+0.096	+0.063	+0.034	+0.010
1.2	+0.283	+0.271	+0.254	+0.234	+0.209	+0.180	+0.142	+0.099	+0.054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

Coefficient of Point

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

Table 2.B
CYLINDRICAL TANKS

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



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

Coefficient of Point

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

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

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

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

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

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

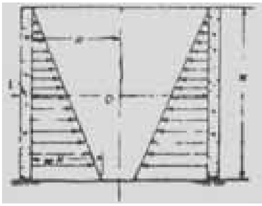
$$At\ x = 0, \qquad -M = EI\frac{d^2v}{dx^2} = 0$$

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

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

Table 3. A
CYLINDRICAL TANKS WITH HINGED BASE AND FREE TOP

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



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

Coefficient of Point

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

3. Cylindrical tank with top slab and fixed base

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

4. Cylindrical tanks with Domes at top and bottom.

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

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

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

Assume thickness of wall = 15 cm.

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

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

Maximum hoop tension occurs at 0.6 *H* from top

Maximum hoop tension

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

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

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

TABLE 4

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

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

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

A_1 = 16°16 cm^2

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

Tensile stress in concrete

$$= \frac{14,736}{100 \times 15 + (m - 1 1) \times 16.1}$$

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

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

Safe

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

= coefficient x wH^3

= 0.0037 x 1000 x 4^3

= + 237.4 kg. m.

Maximum – ve *B.M.* (tension inside)

= 0.01436 x 1000 x 4^3

= -919.1 kg. m.

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

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

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

Minimum percentage of steel to be provided

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

= 0.3 – 0.014 = 0.286.

Minimum steel area required

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

Minimum reinforcement on one face = 2.145 cm^2

Provide 8 mm ϕ bars at 20cm. centers.

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

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

Provide 12 mm bars at 11 cm centers.

Shear force

Maximum shear at base of wall

$$= 0.1724\ wH^2$$

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

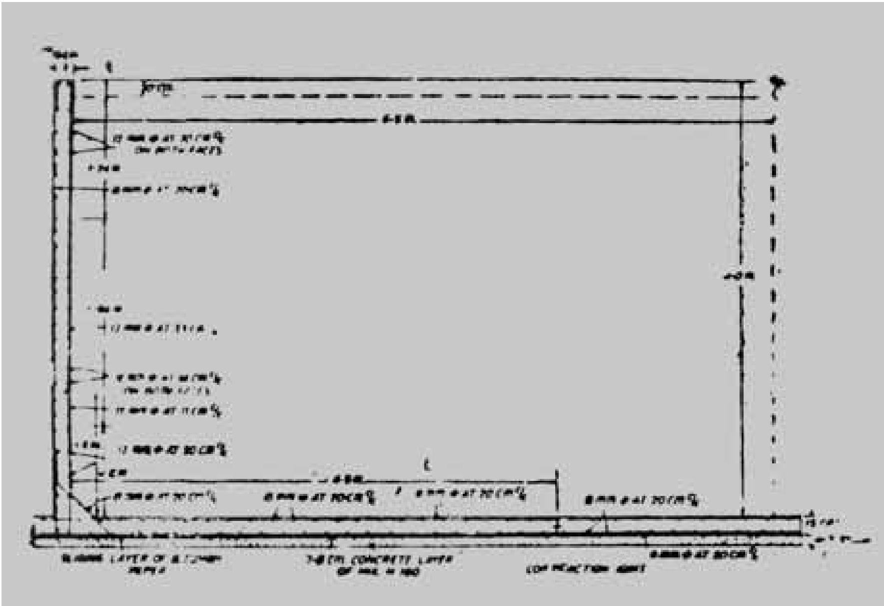


Fig. 7

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

Safe.

Check for Bond

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

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

Safe.

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

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

between 12 to 30, the cantilever portion may be assumed as H/4 or 1 m from base, whichever is more.

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

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

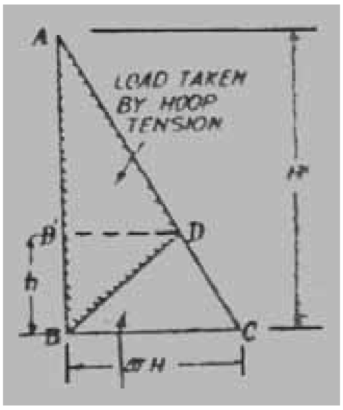


Fig.8

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

Sol. D = 13 m H = 4 m

Assume thickness of wall t = 15 cm

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

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

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

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

= 17,333 kg. per meter height.

Area of steel required

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

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

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

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

Effective depth = 15-4 = 11 cm

Area of steel required

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

Provide 12 mm ϕ bars at 8 cm center.

Distribution steel

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

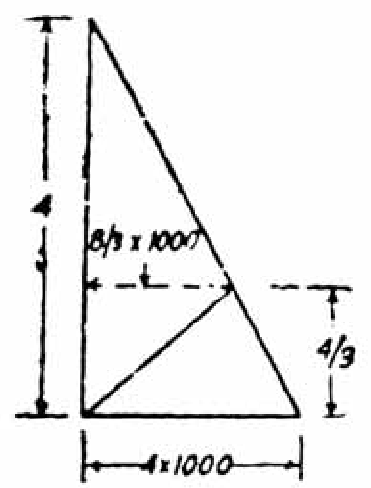


Fig. 9

Steel area = $\frac{0.286}{100} \times 15 \times 100$

= 4.29 cm^2

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

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

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

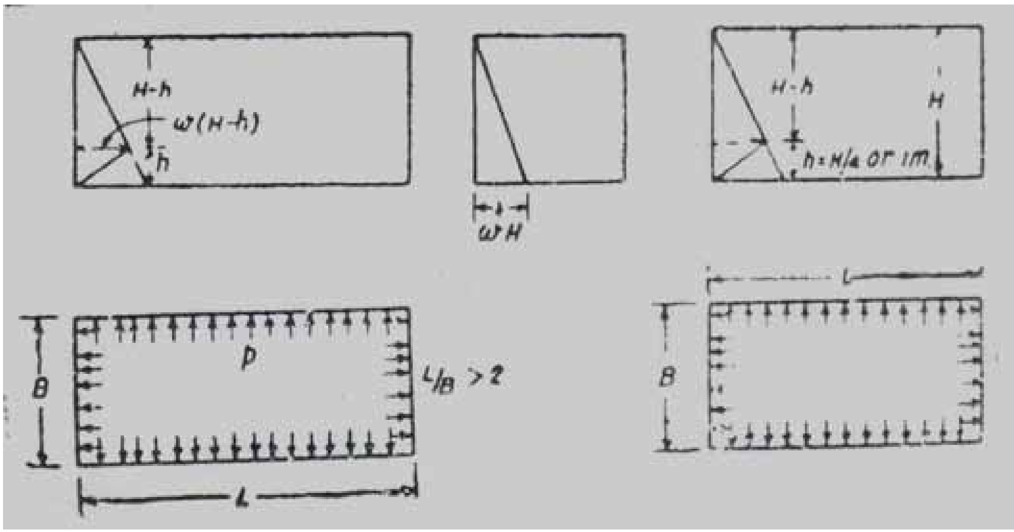
For rectangular tanks in which ratio of length to breadth is less than 2, tank walls are designed as continuous frame subjected to pressure varying from zero at to top to maximum at H/4 or 1 m., from base, whichever is more. The bottom portion H/4 (or) 1 m whichever is more is designed as cantilever.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

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

and direct tension in short walls

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

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



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

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

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

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

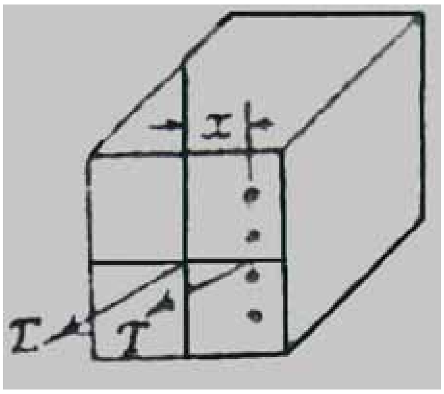


Fig. 10 (c)

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

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

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

Effective height = 335 cm.

Volume of tank to be required

= 80,000 liters = 80,000,000 c.c

Area of tank to be provided

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

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

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

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

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

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

Pressure at depth of 2.5 m.

p = wh = 2.5 x 1000

= 2,500 kg/m^2

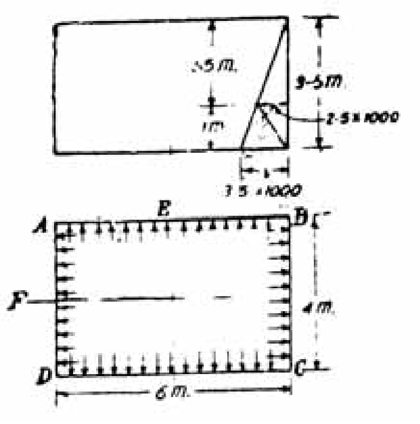


Fig. 11

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

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

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

B.M. at center of long span

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

= 5417 kg. m.

B.M. at center of shorter span

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

= 5000 – 5833 = - 833 kg. m.

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

Effective depth required

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

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

Direct tension in long wall

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

Direct tension in the short wall

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

Design of section

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

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

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

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

$$Q = ½\ ckj$$

$$= ½ \text{ x } 70 \text{ x } 0.48 \text{ x } 0.84 = 14.11$$

Considering effect of bending only, effective depth required

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

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

Net moment = $M - T$ x x

Area of steel

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

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

= 29.77 + 50 = 34.77 cm^2.

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

Steel at center of span

Tension occurs away from water face, c = 70, t = 1250, m = 13, n = 0.42d, j = 0.86d, Q = 12.64

Area of steel

$$= \frac{541{,}700 - 5000(21.5 - 12.5)}{0.86 \times 21.5 \times 1250} + \frac{5000}{1000}$$

= 21.49 + 5.0

= 26.49 cm^2.

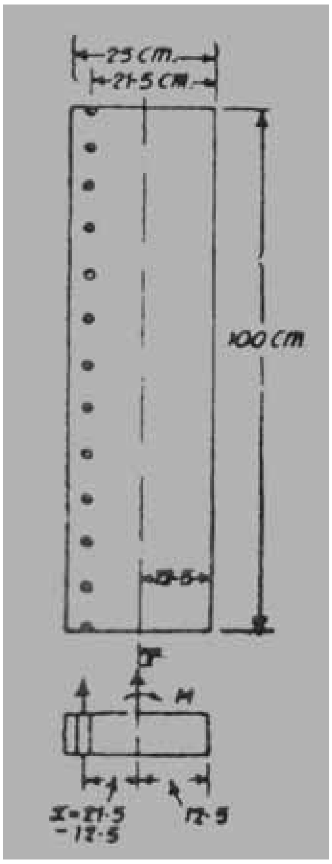


Fig. 12

Half the bars from inner face at support are bent in outer face providing area of $\frac{39.28}{2}$ = 19.64 cm^2.

Remaining area to be provided

$$= 26.49 - 19.64 = 6.85 \text{ cm}^2.$$

Additional reinforcement of 16 mm. ϕ bars at 16 cm. centers is provided.

At the center of short span *B.M.* is of negative sign, however nominal reinforcement is provided on that face.

Cantilever Moment

Cantilever moment = 3.5 x 1000 x ½ x 1 x $\frac{1}{3}$ = 583.3 kg.m.

Area of steel required

$$= \frac{583.3 \times 100}{0.84 \times 21.5 \times 1000}$$

$$= 3.23 \text{ cm}^2.$$

Provide 8 mm. ϕ bars at 15 cm. centers.

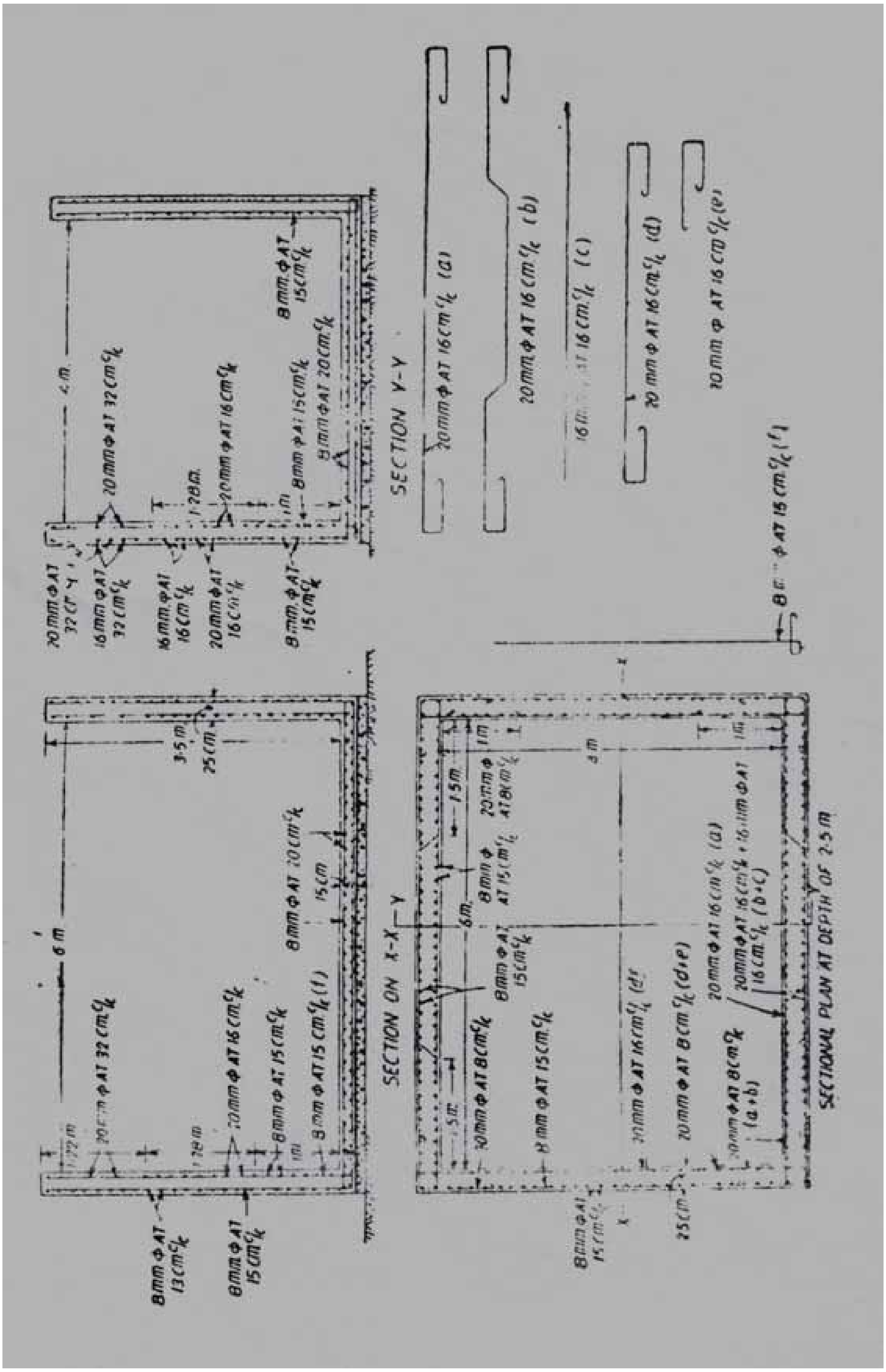


Fig. 13

Distribution steel

Distribution steel = 0.3 – $\frac{0.1\times(25-10)}{(45-10)}$

= 0.3 – 0.043

= 0.257 %

Area of steel = $\frac{0.257}{100}$ x 25 x 100

= 6.425 cm^2.

Area on each face = 3.212 cm^2.

Provide 8 mm ϕ bars at 15 cm. centers.

Base Slab

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

10. Underground Tanks. The design principles of underground tanks are same as for tanks resting on the ground. The walls of the underground tanks are subjected to internal water pressure and outside earth pressure. The section of wall is designed for water pressure and earth pressure acting separately as well as acting simultaneously.

Whenever there is possibility of water table to rise, soil becomes saturated and earth pressure exerted by saturated soil should be taken into consideration.

Ex. 8.5. Design an underground reservoir 12 m. x 4 m. x 4 m. deep. The long walls will be designed as cantilevers and the top portion of the short walls will be designed as slab supported by long walls. Bottom one metre of short walls will be designed as cantilever slab.

Design of long wall

1. **Pressure of saturated soil acting from outside and no water pressure from inside.** Earth pressure at base will be due to water pressure plus due to submerged weight of soil.

p = 1000 x 4 + (1600 – 1000) x ½ x 4

= 4000 + 800 = 4800 kg/m^2.

Maximum *B.M.* at base of long wall

= 4800 x $\frac{4}{2}\times\frac{4}{3}$

= 12,800 kg.m.

= 1,280,000 kg. cm.

Effective depth required

$$= \sqrt{\frac{12.80,000}{14.11 \times 100}}$$

$$= 30.11 \text{ cm.}$$

Provide overall depth of 35 cm. with effective depth of 31 cm.

$$\text{Area of steel} = \frac{1,210,000}{0.84 \times 1000 \times 31}$$

$$= 48.2 \text{ cm}^2.$$

Provide 20 mm ϕ bars at 6 cm. centers on outside face. Area of steel provided = 52.36 cm^2.

Reinforcement is curtailed in the same manner as in the second case.

Direct compression in long walls. Direct compression is caused in long walls because of earth pressure acting on short walls which act as slab supported on long walls.

Direct compression at 1 m above base

$$= 3600 \text{ x } \frac{4}{2}$$

$$= 7200 \text{ kg.}$$

This will be taken by wall and the distribution steel provided.

2. **Water pressure acting from inside and no earth pressure acting from outside.**

Maximum water pressure at base

$$= 1000 \text{ x } 4 = 4000 \text{ kg/m}^2$$

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

$$= \frac{32,000}{3} = 10,667 \text{ kg.m}$$

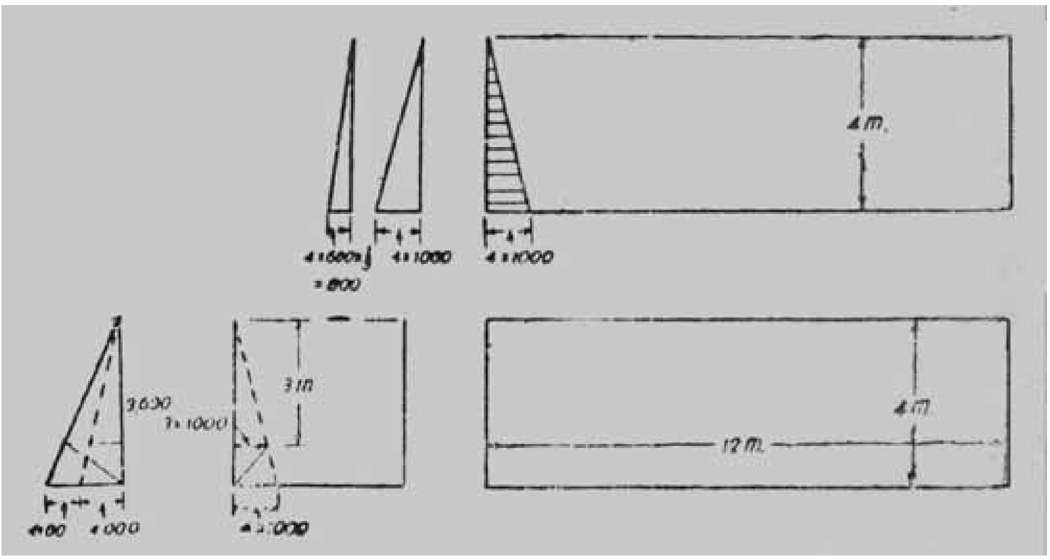


Fig. 14

Area of steel required

$$= \frac{10.667 \times 100}{0.84 \times 1000 \times 31}$$

$= 40.85\ cm^2$.

Provide 20 mm ϕ bars at 7 cm. centers on inside face. Area of steel provided.

$= 44.86\ cm^2$.

Curtailment of reinforcement. Let A_{th} be the reinforcement required at depth *h*. The *B.M.* at any depth is proportional to h^2.

$$\therefore \qquad \frac{A_{th}}{A_t} = \frac{h^3}{H^3}$$

$$h = \sqrt[3]{\frac{A_{th}}{A_t}} \times H$$

Depth where half the bars can be curtailed, $A_{th} = ½\ A_t$.

$$\therefore \qquad h = \sqrt[3]{½} \times 4$$

$= 3.175$ m

Taking bond length as 20d, the bars are curtailed at 1.25 from base. 20 mm ϕ at 12 cm. *c/c* are provided on outside and 20 mm ϕ at 14 cm. *c/c* inside.

Depth where only ¼th reinforcement is required $A_{th} = ¼A_t$.

$$h = \sqrt[3]{½} \times 4$$

$= 2.52$ m

¼th reinforcement i.e. 20 mm ϕ at 24 cm. centers are provided on outside and 20 mm ϕ at 28 cm. *c/c* inside at 2 m. and above from base.

Distribution steel

% of distribution steel

$$= 0.3 - \frac{0.1 \times (35 - 10)}{(45 - 10)}$$

$= 0.3 – 0.06 = 0.24$

$$\text{Area of steel} = 0.24 \times \frac{35 \times 100}{100}$$

$= 7.68\ cm^2$.

Area to be provided on each face

$= 3.84\ cm^2$.

Provide 8 mm ϕ bars at 13 cm centers.

Direct tension in long walls. Direct tension is caused in long walls because of water pressure acting on short walls which act as slab supported on long walls.

Direct tension at 1 m above base

$$= 3 \text{ x } 1000 \text{ x } \frac{4}{2} = 6000 \text{ kg.}$$

Area of steel required

$$= \frac{6000}{1000} = 6 \text{ cm}^2.$$

Area of distribution steel provided is 7.6 cm^2.
Distribution steel will take direct tension.

Design of short wall

1. Water pressure acting from inside and no earth pressure acting from outside.

Bottom one metre acts as cantilever and remaining 3 m acts as slab supported on long walls.

Water pressure at depth of 3 m

$$= 1000 \text{ x } 3 = 3000 \text{ kg/m}^2.$$

Maximum *B.M.* is assumed as

$$= \frac{wl^2}{12} \text{ at supports.}$$

$$B.M. \text{ at support} = \frac{wl^2}{12} = \frac{3000 \times 4^2}{12}$$

$$= 4000 \text{ kg.m}$$

Direct tension from end one metre of long wall

$$= 3000 \text{ x } 1 = 3000 \text{ kg.}$$

$$\text{Net } B.M. = M - T \text{ x } x = 4000 - 3000 \text{ x } 0.12$$

$$= 3640 \text{ kg.m.}$$

$$\text{Area of steel} = \frac{3640 \times 100}{0.84 \times 31 \times 1000} + \frac{3000}{1000} \text{ (Tension inside)}$$

$$= 13.97 + 3$$

$$= 16.97 \text{ cm}^2.$$

Provide 12 mm ϕ bars at 6 cm centers in one metre length. As the bending moment is proportional to the depth of water, reinforcement will vary linearly with depth of water. At depth of 2 m., 12 mm bars at 12 cm. centers are provided. At depth of 1 m., 12 mm ϕ bars at 18 cm centers are provided.

B.M at center of span

$$= \frac{wl^2}{12} = \frac{3000 \times 4^2}{16}$$

= 3000 kg m.

Net $B.M.$ = $M - T$ x x = 3000 – 3000 x 0.12

= 2640 kg. m.

$$\text{Area of steel} = \frac{2640 \times 100}{0.86 \times 31 \times 1000} + \frac{3000}{1000}$$

= 10.14 + 3 = 13.14 cm^2

Actually the section is doubly reinforced as steel is to be proded on front face for the bending moment produced due to outside pressure.

12 mm ϕ bars are provided at 6 cm centers.

A_t provided = 18.85 cm^2.

The reinforcement is reduced linearly towards top. At depth of metres 12 mm ϕ bars are provided at 12 cm. centers. At depth of 1 metre 12 mm ϕ bars are provided at 18 cm. centers.

Design of bottom one metre

$B.M.$= 1000 x 4 x ½ x ½

= 667 kg.m

$$\text{Area of steel} = \frac{667 \times 100}{0.84 \times 31 \times 1000}$$

= 2.56 cm^2

Minimum reinforcement of 8 mm ϕ at 13 cm centers is provided.

Distribution steel of 8 mm ϕ at 13 cm centers is provided on both faces.

2. Pressure of Saturated soil acting from outside and no water pressure from inside.

Direct compression due to cantilever action of one metre length of long wall = 3600 kg.

Pressure p = 3600 kg/ m^2

$$B.M. \text{ at support} = \frac{pl^2}{12} = \frac{3600 \times 4 \times 4}{12}$$

= 4800 kg. m

$$\text{B.M at center of span} = \frac{3600 \times 4 \times 4}{12} = 3600 \text{ kg.m}$$

Area of steel required, considering bending only,

$$A_t = \frac{4800 \times 100}{0.86 \times 1000 \times 31}$$

$= 18.43\ cm^2$ (Tension outside)

Provide 12 mm ϕ bars at 6 cm centers,

A_t provided = $18.85\ cm^2$

Actually the section is doubly reinforced and area of steel required will be less. The effect of direct compressive force is not considered as it is very small and compressive reinforcement is provided.

Area of steel at center of span

$$= \frac{3600 \times 100}{0.84 \times 1000 \times 31}$$

$= 13.81\ cm^2$.

Provide 12 mm ϕ bars at 6 cm centers.

Design of bottom one metre

B.M = ½ x 4,800 x ½

= 800 kg.m

Area of steel $\frac{800 \times 100}{0.84 \times 1000 \times 31}$ $= 3.07\ cm^2$.

Provide 8 mm ϕ bars at 13 cm centers.

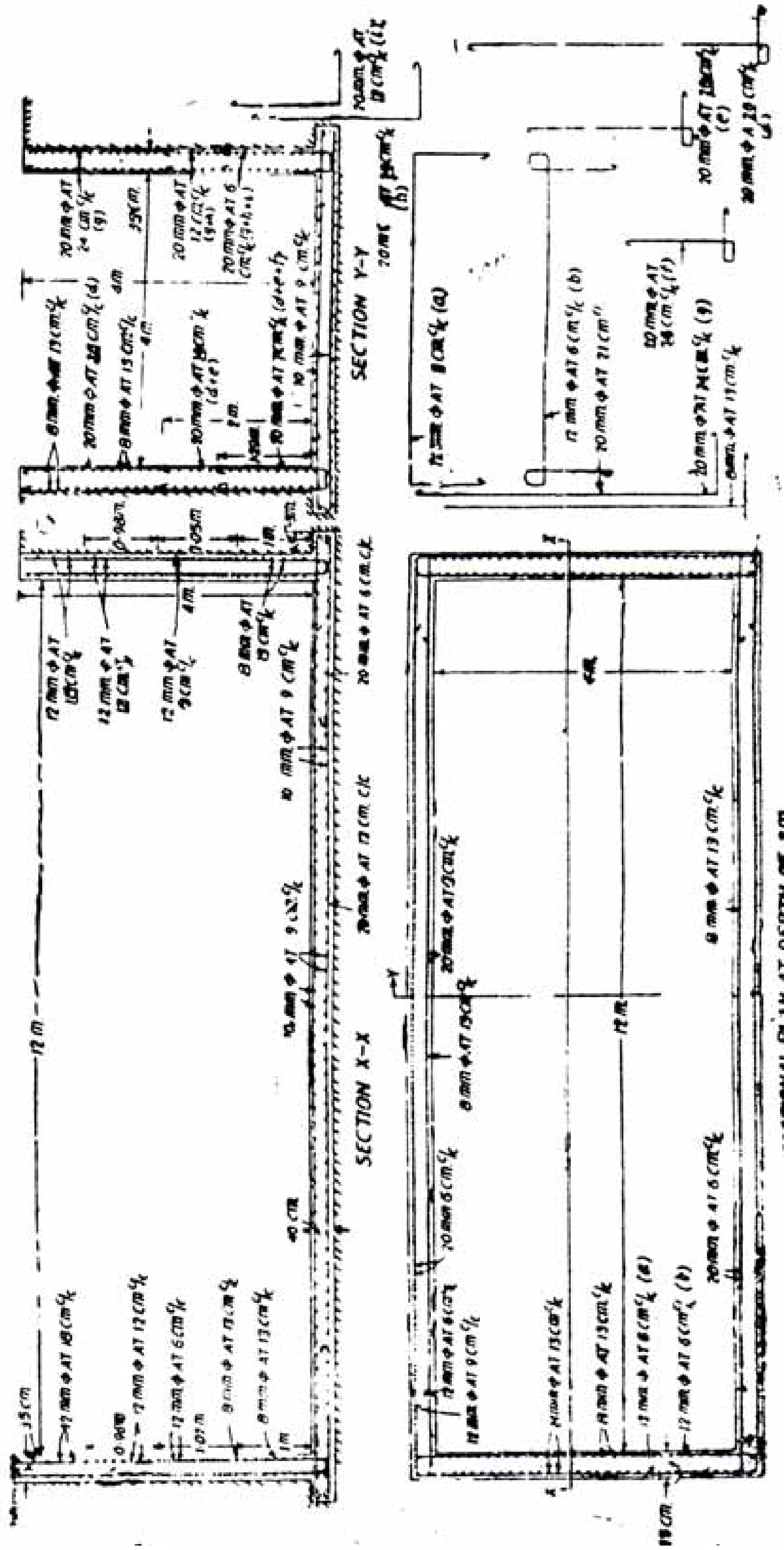


Fig. 15

Base slab

Check Against Uplift

Projection of 0.3 m is provided beyond the face of the walls to add to stability against uplift.

Weight of vertical walls

Long walls = 2 x 12.7 x 0.35 x 4 x 2400 = 85,344 kg.

Short walls = 2 x 4 x 4 x 0.35 x 2400 = 26,880 kg.

Base slab = 13.3 x 5.30 x 0.4 x 2400 = 67,674 kg.

Weight of earth on projection = 2(13.3 + 4.7) x 4 x 0.3 x 1600

= 69,120 kg.

Uplift pressure due to pressure of water at bottom of tank

= 5.3 x 13.3 x 4.4 x 1000

= 249,014 kg.

Total downward weight

= 85,344 + 26,880 + 67,670 + 69,120

= 249,014 kg.

Frictional resistance required

= 310,200 – 249,012

= 61,188 kg.

Pressure of submerged earth and water at depth of 4.4 m.

$$= 1000 \text{ x } 4.4 + \frac{1}{3}(1600\text{-}1000) \text{ x } 4.4$$

= 4400 + 880 = 5280 kg/m^2

Total pressure per one metre length of walls

= ½ x 4.4 x 5280

= 11,616 kg.

As the soil is saturated, the angle of friction of submerged soil will be low. Assuming coefficient of friction as 0.15, frictional resistance per metre length of wall

= 0.15 x11616

= 1742.4 kg.

Total frictional resistance of four sides

= 2(5.3 + 13.3) x 1742 .4

= 64,820 > 61,188 Safe.

Design of Base Slab

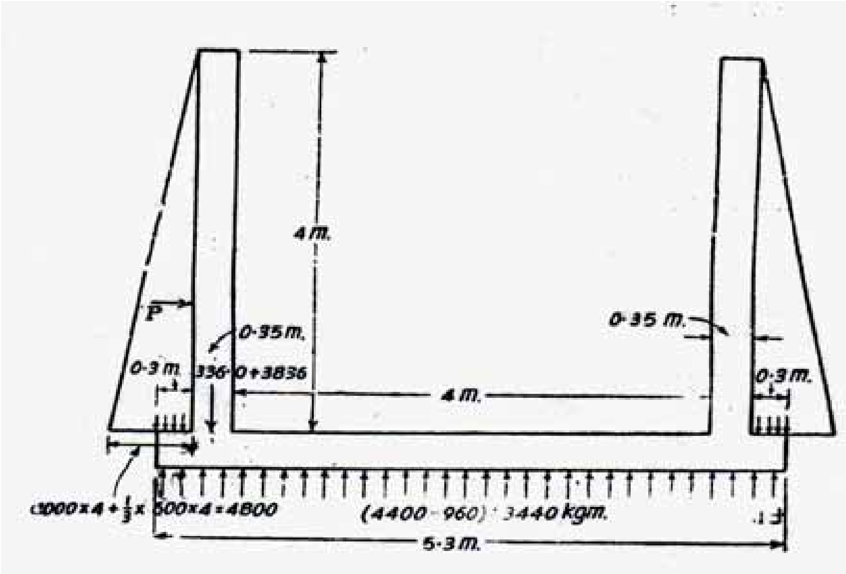


Fig. 16

Consider one metre length of slab.

Upward pressure of water per sq. metre

$= 4.4 \times 1000 = 4400 \text{ kg/m}^2.$

Self weight of slab $= 1 \times 1 \times 0.4 \times 2400 = 960 \text{ kg/m}^2.$

Net upward pressure $= 4400 - 960 = 3440 \text{ kg/m}^2.$

Weight of wall per metre run

$= 0.35 \times 4 \times 2400 = 3360$ kg.

Weight of earth on projection

$= 1600 \times 4.0 = 6400 \text{ kg/m}^2.$

Net unbalanced force

$= 3440 \times 5.3 - 2(3360 + 6400 \times 0.3)$

$= 18{,}232 - 10{,}560$

$= 7672$ kg.

Reaction on each wall

$$= \frac{7672}{2} = 3836 \text{ kg.}$$

B.M. at edge of cantilever portion

$$= \frac{3440 \times 0.3^2}{2} + 4800 \text{ x } \frac{4}{2}\left(\frac{4}{3} + 0.2\right) - 6400 \text{ x } \frac{0.3^2}{2}$$

= 154.8 + 14,720 – 288
= 14,586.8 kg. m.

B.M at center of span

$$= \frac{3440}{2} \times \left(\frac{5.3}{2}\right)^2 + 4800 \times \frac{4}{2}$$

$$\times \left(\frac{4}{3} + 0.2\right) - (3360 + 3836)$$

$$\left(\frac{5.3}{2} - 0.3 - \frac{0.35}{2}\right) \text{-6400 x 0.3}$$

$$\times \left(\frac{5.3}{2} - 0.15\right)$$

= 12,080 + 14,720 – 15,650 – 4,800
= 6530 kg.m.

Effective depth required

$$= \sqrt{\frac{14586.8 \times 100}{14.11 \times 100}}$$

= 32.15 cm

Provide overall depth of 40 cm. with effective depth of 35 cm.

Area of steel required at support

$$= \frac{14{,}586.8 \times 100}{0.84 \times 1000 \times 35}$$

= 49.58 cm^2.

Provide 20 mm ϕ at bars at 12 cm. centers.

Reinforcement at centre of span

$$= \frac{6350 \times 100}{0.84 \times 1000 \times 35}$$

= 21.60 cm^2.

Provide 20 mm ϕ bars at 12 cm. centres.

Minimum reinforcement

$$= 0.3 - 0.1 \text{ x } \frac{30}{35}$$

= 0.3 – 0.09 = 0.21

Reinforcement $= \frac{0.21}{100} \times 40 \times 100$

$= 8.4 \text{ cm}^2$

Provide 10 mm ϕ bars at 9 cm .*c/c* as distribution steel at bottom and 10 mm ϕ bars at 9 cm. *c/c* both ways at top.

Maximum *S. F* at edge

$= 3360 + 3836 + 6400 \times 0.3 - 3440(0.3 + 0.35)$

$= 3360 + 3836 + 1920 - 2236$

$= 6880$ kg.

Shear stress $= \frac{6880}{0.84 \times 100 \times 35}$

$= 2.24 \text{ kg/cm}^2$. Safe.

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

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

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

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

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

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

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

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

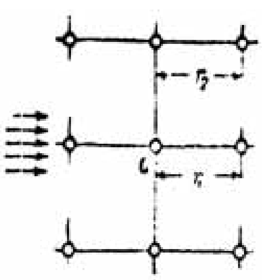


Fig. 17

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

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

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

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

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

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

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

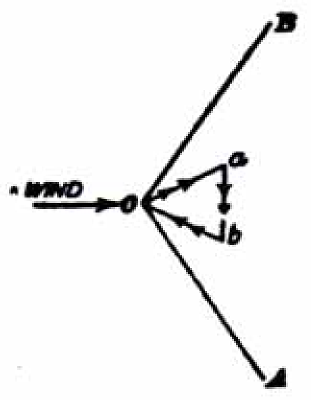


Fig. 18

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

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

Radius of dome is given by

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

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

Thickness of 10 cm. is provided for dome.

Self-load per square metre

= 10 x 24 = 240 kg.

Live load per square metre

= 150 kg.

Total load = 390 kg/m^2.

Hoop stress at any angle θ is given by

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

Meridional stress

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

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

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

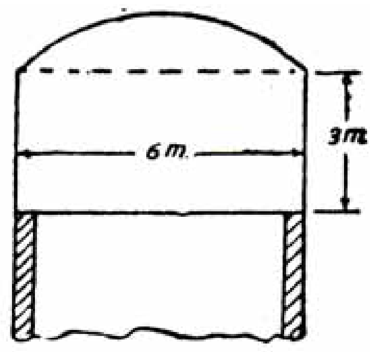


Fig. 9 (a)

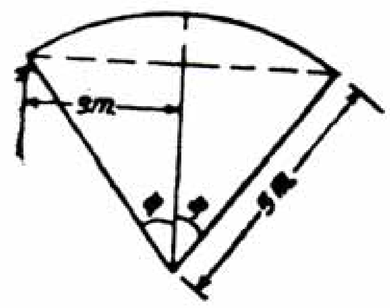


Fig. 9 (b)

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

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

Hoop stress = $\dfrac{390 \text{ x } 5\,(\cos^2\theta + \cos\theta - 1)}{0.1(1+\cos\ \theta)} x 10^{-4}\, kg\ /\text{cm}^2$

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

Maximum meridional stress occurs at

$\theta = \phi = 36°52'$.

Meridional stress = 1.95 x $\dfrac{(1-0.8)}{0.6 \text{ x } 0.6}$

$= 1.08$ kg/ cm^2.

Maximum hoop stress occurs at

$\theta = 0°$

Hoop stress = 1.95 x $\dfrac{(1+1-1)}{1+1}$

= 0.975 kg/cm^2.

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

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

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

= 1.08 x 10 x 1 = 10.8 kg

Horizontal component of meridional thrust

= 10.8 cos ϕ

= 10.8 x 0.8.

Hoop tension = $\dfrac{pd}{2} = 10.8 \text{ x } 0.8 \text{ x } \dfrac{600}{2}$

= 2592 kg.

Area of steel required

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

Provide 4 bars of 10 mm ϕ. Area provided

= 3.14 cm^2.

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

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

Stress in concrete. Equivalent area

= 20 x 15 + 12 x 3.14

= 300 + 37.68

= 337.68 cm^2.

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

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

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

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

From table 17.2.

Maximum hoop tension

= 0.608 wHR

= 0.608 x 1000 x 3 x 3

= 5472 kg.

Maximum hoop tension occurs at depth of

0.6 H = 1.8 m. from top.

Maximum - ve *B.M.* = 0.0122 wH^2

= 0.0122 x 1000 x 27

= 329.4 kg. m.

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

Maximum +ve *B.M.* = 0.0029 x 1000 x 27

= 78.3 kg. m.

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

0.7 H = 2.1 m. from top.

Maximum shear at base

= 0.158 wH^{2ss}

= 0.158 x 1000 x 9

= 1422 kg.

Steel required for hoop tension

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

= 5.472 cm^2.

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

Minimum steel required

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

= 0.286%

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

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

Steel required for - ve *B.M.*

Effective depth = 15 - 4 = 11 cm.

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

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

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

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

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

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

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

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

Maximum radial negative moment

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

Effective depth required

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

= 16.8 cm

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

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

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

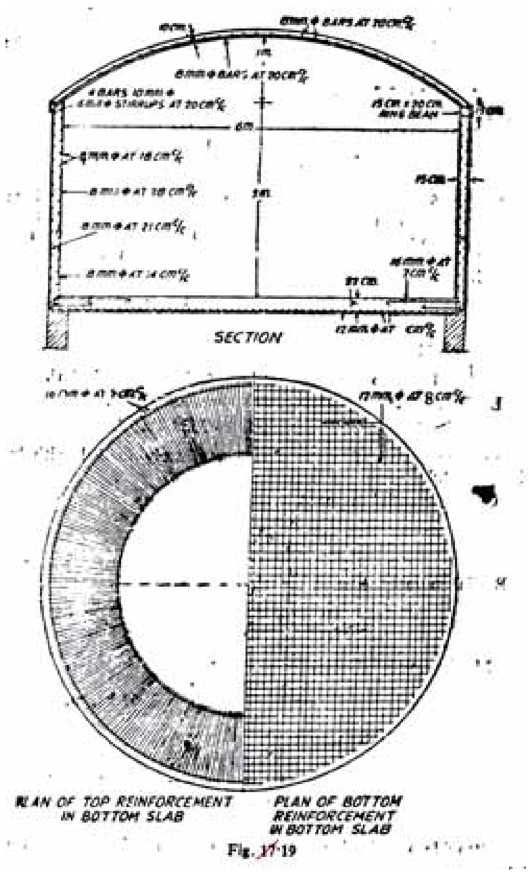


Fig. 19

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

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

Circumferential moment

$= 1984.5$ kg. m.

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

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

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

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

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

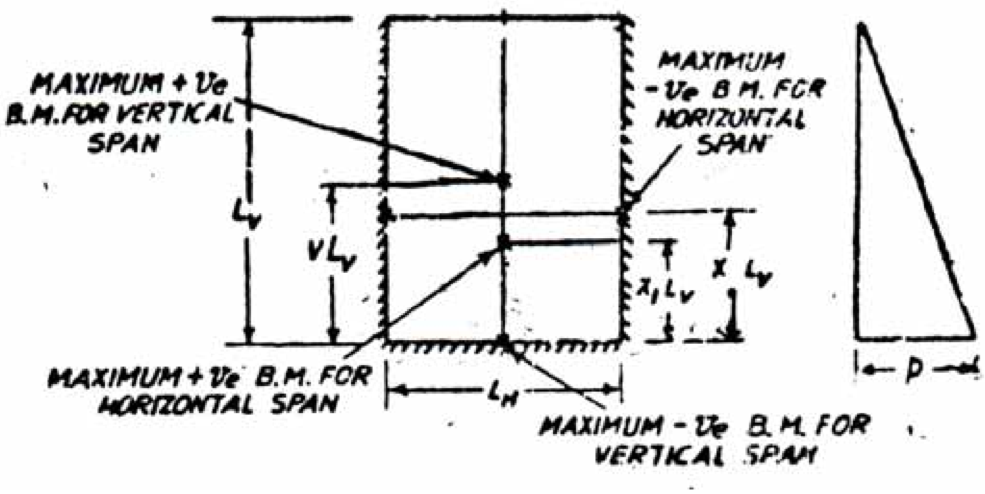


Fig. 20

B.M. = kpl^2

where l is the span,

k is coefficient given in Table 5

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

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

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

Table 5

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

For $\frac{L_H}{L_V} < 0.5$, whole load will act along the horizontal span and maximum -ve

$B.M. = \frac{pL_H^2}{12}$ and maximum + ve $B.M. = \frac{pL^2_H}{16}$

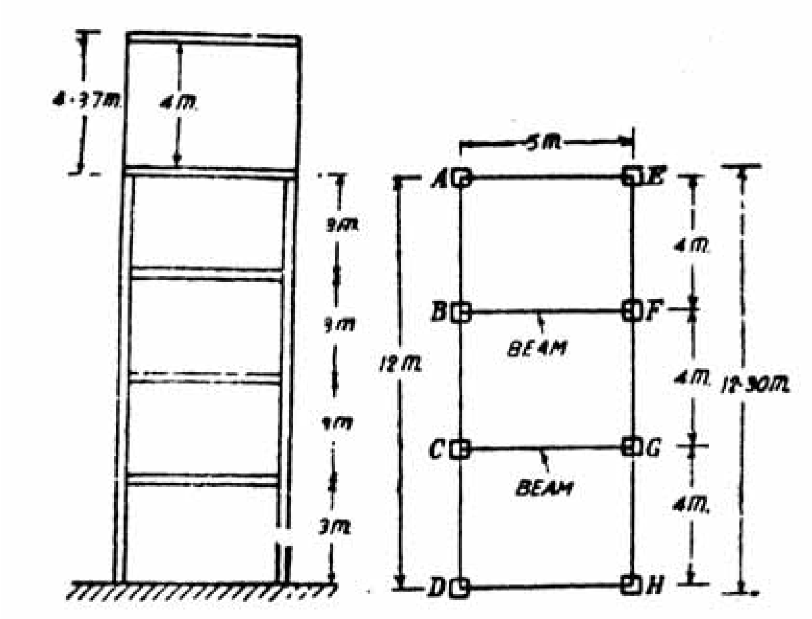


Fig. 21

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

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

Concrete mix used is *M* 200.

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

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

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

Maximum *B.M.* $= \frac{wl^2}{8} = \frac{510 \text{ x } 5^2}{8}$

= 1593.75 kg. m.
= 159,375 kg. cm.

For mix. *M* 200, C = 70, t = 1400, m = 13, Q = 12.10.

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

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

$$A_t = \frac{159{,}375}{1400 \text{ x } 0.87 \text{ x } 13}$$

$= 10.07 \text{ cm}^2$.

Provide 12 mm ϕ bars at 11 cm. centre.

Distribution steel $= \dfrac{0.15 \times 15 \times 100}{100}$

$= 2.25 \text{ cm}^2$.

Provide 6 mm ϕ bars at 12 cm centres.

Design of Long Walls. As the length of long wall is greater than twice the height of the wall, the wall will be designed as spanning vertically,

p = 4 x 1000 = 4000 kg/m^2

Maximum *B.M.* at base

$$= \frac{pl^2}{15} = \frac{4000 \text{ x } 4^2}{15} = \frac{64{,}000}{15}$$

= 4266.66 kg. m. = 426,666 kg. cm.

The tension will be on water side

$c = 70$, $t = 1000$, $Q = 14.11$, $j = 0.84$.

Effective depth $= \sqrt{\dfrac{426.666}{14.11 \text{ x } 100}}$. = 17.39 cm

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

Area of steel required

$$= \frac{426666}{0.84 \text{ x } 1000 \text{ x } 18} = 28.05 \text{ cm}^2.$$

Provide 20 mm. ϕ bars at 11 cm. centres.

A provided = 28.56 cm^2.

Maximum + ve *B.M.*

$$= \frac{pLv^2}{33.5} = \frac{4000 \text{ x } 4^2}{33.5}$$

= 1911 kg. m.

= 191,100 kg. cm.

Area of steel required $= \dfrac{191{,}100}{0.84 \text{ x } 1000 \text{ x } 18} = 12.60 \text{ cm}^2$.

Provide 12 mm ϕ bars at 9 cm. centres.

Distribution Steel

Distribution steel $= \left(0.3 - \frac{0.1 \times 12}{35}\right)$ % = 0.266 %

Steel area $= \frac{0.266 \text{ x } 22 \text{ x } 100}{100} = 5.852 \text{ cm}^2$.

Half the reinforcement is provided on each face. Provide 8 mm ϕ at 17 cm. centres on each face. There will be direct load on the long wall due to self weight of the roof. It is assumed that whole load from roof is transferred to long walls.

Load per metre length of wall

$= 510 \text{ x } \frac{5}{2} + 528 \text{x } 4$

= 1275 + 2112 = 3,387 kg.

Direct load is very small, its effect is neglected. Actually reinforcement is provided on both faces vertically as well as horizontally,

There will be direct tension in long walls due to pressure on short walls but its effect is negligible.

Design of Short Walls

$$\frac{L_H}{L_V} = \frac{5}{4} = 1.25.$$

Coefficients for *B.M.* are obtained from Table 17.5 by linear interpolation.

p = 4 x 1000 = 4000 kg/m^2.

Maximum - ve *B M.* for vertical span

= 0.0345 x 4000 x 4^2

= 2208 kg. m.

Maximum + ve *B.M.* for vertical span

= 0.02 x 4000 x 4^2

= 1280 kg. m.

Maximum -ve *B.M.* for horizontal span

= 0.0195 x 4000 x 5^2

= 1950 kg. m.

Maximum +ve *B.M.* for horizontal span

= 0.008 x 4000 x 5^2

= 800 kg m.

Overall thickness of 22 cm. is provided.

Steel in Vertical Span

For -ve *B.M.* $= \frac{2208 \text{ x } 100}{0.84 \text{ x } 18 \text{ x } 1000}$ [tension on water face]

= 14.6 cm^2.

Provide 12 mm. ϕ bars at 7cm. centre

For + ve *B.M.* $= \dfrac{1280 \text{ x } 100}{0.84 \text{ x } 18 \text{ x } 1000}$ [tension outside]

= 8.40 cm^2.

Provide 10 mm. ϕ bars at 9 cm. centres.

Steel in Horizontal Span.

For -ve *B.M.* $= \dfrac{1950 \text{ x } 100}{0.84 \text{ x } (18 - 0.6 - 0.6) \text{ x } 1000}$

[tension on water side]

= 14.0 cm^2.

Provide 12 mm. ϕ bars at 8 cm. centres.

For + ve *B.M.* $= \dfrac{800 \times 100}{0.84 \times 16.8 \times 1000}$ (tension outside)

= 5.62 cm^2.

Provide 12 mm. ϕ bars at 16 cm. centres.

There will be small amount of direct tension in short walls due to water pressure on long walls but its effect will be very small and is neglected.

Design of base slab The slab is built monolithic with beam on all the four edges. For findin bending moment coefficients it will be considered continuous on all four edges.

Bending moment coefficients are obtained by interpolation from Table 8.5

$$\frac{L_H}{L_V} = \frac{5}{4} = 1.25.$$

Assume 22 cm. thick slab.

Self load = 0.22 x 1 x 1 x 2400 = 528 kg/m^2.

Load of water 4 x 1000 = 4000 kg/m^2.

Total load = 4528 kg/m^2.

***B.M.* in Short Span**

-ve *B.M.* at continuous edge

= 0.0475 x 4528 x 4^3

= 3985 kg m.

+ ve *B.M.* at mid-span

= 0.036 x 4528 x 4^2

= 2610 kg. m.

***B. M.* in Long Span**

-ve *B.M.* at continuous edge

= 0.033 x 4528 x 4^2

= 2390 kg. m.

+ve *B.M.* at mid - span

= 0.025 x 4528 x 4^2

= 1812 kg. m.

Maximum *B.M.* = 3985 kg.m. (tension on water side0

c = 70, t = 1000, Q = 14.11, n = 0.48, j = 0.84

Effective depth required

$$= \sqrt{\frac{3985x100}{14.11x100}} = 16.85 \text{ cm.}$$

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

Steel in Short Span

Steel required for -ve *B.M.* at continuous edge

$$= \frac{3985x100}{0.84x1000x18}$$

$$= 26.3 \text{ cm}^2.$$

Provide 16 mm. ϕ bars at 7 cm. centres.

*A*t provided = 28.73 cm^2.

Steel for +ve *B.M.* $= \frac{2610 \times 100}{0.84 \times 1000 \times 18} = 17.28 \text{ cm}^2.$

Provide 16 mm. ϕ bars at 11 cm. centres.

Steel in Long Span

Effective depth = 18-0.8-0.8 = 16.4 cm.

Steel for +ve *B.M.* $= \frac{2390 \times 100}{0.84 \times 1000 \times 16.4} = 17.28 \text{ cm}^2.$

Provide 16 mm. ϕ bars at 11 cm. centres.

$A_t = 18.27 \text{ cm}^2.$

Steel for + ve *B.M.* $= \frac{1812 \times 100}{0.84 \times 1000 \times 16.4}$

$= 13.0 \text{ cm}^2.$

Half the bars from support are bent to the mid-span giving 16 mm. ϕ at 22 cm. centres and steel area of $\frac{18.27}{2}$.

Remaining steel required = $13.0 - \frac{18.27}{2} = 3.86 \text{ cm}^2.$

10 mm. ϕ bars at 22 cm. centres are provided.

Design of Beams

Design of Beams B_1

Load from slab = w = 4528 kg/m^2.

Self weight = 0.48 x 0.3 x 2400

= 345.6 kg/m.

The loading on the beam is shown in

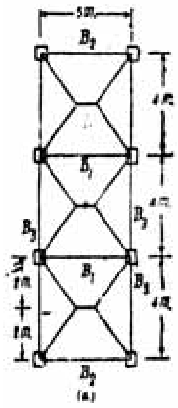


Fig. 22 (a).

As the beam is built monolithic with columns, partial fixity is assumed at the ends. *B.M*. taken for design purpose will be 0.8 times simply supported bending moment.

$$\text{Max. } B.M. = 0.8 \left[6w \times 2.5 - \frac{4w}{2} \times 2 \left(\frac{2}{3} + 0.5 \right) - 4w \times 0.5 \pm \times 0.25 + \frac{345.6 \times 5^2}{8} \right]$$

$$= 0.8 \left[15w - 4w \times \frac{7}{6} - 0.5\ w + 1080 \right]$$

= 0.8 [9.83w + 1080]

= 0.8 [9.83 x 4528 + 1080]

= 36,472 kg. m.

= 3,647,200 kg. cm.

Concrete mix used is *M* 200.

The beam is designed as *T*-beam.

Effective depth = 70-8 = 62 cm.

Assume lever arm = $62 - \frac{22}{2}$

` = 51 cm.

Area of steel = $\frac{3,647,200}{51 \text{x} 1400}$

= 51.2 cm^2.

Provide 4 bars of 32 mm. ϕ and 4 bars of 25 mm. ϕ .

Steel area provided = 38.17 + 19.63

= 51.80 cm^2.

Check for Stress

Flange width will be least of

(i) Centre to centre = 200 cm.

(ii) $^1/_3$ rd span $\dfrac{500}{3}$ = 167

(iii) 12 $d_s + b_r$ = 12 x 22 + 30 = 294 cm.
Flange width = 167 cm.

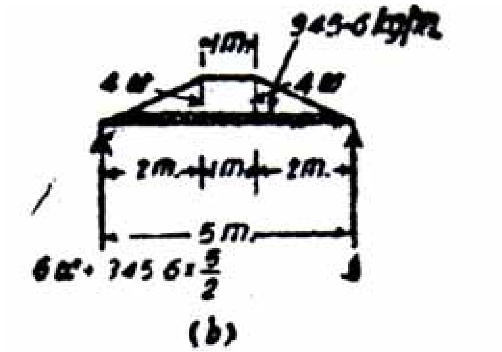


Fig. 22 (b)

Assume neutral axis to lie in the flange.

$167 \times \dfrac{n^2}{2} = 13 \times 51.8\,(62-n)$

$n^2 + 8.06n = 500$

$(n + 4.03)^2 = 500 + 16.24$

$= 516.24$

$\therefore\; n = 22.72 - 4.03$

$= 18.69$ cm.

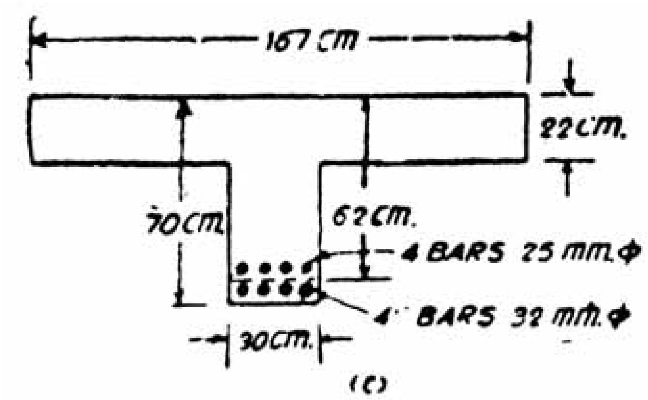


Fig 22 (c)

Maximum stress in steel

$$= \frac{3{,}647{,}200}{\left(62 - \dfrac{18.69}{3}\right) \times 51.8}$$

$= 1263$ kg/cm^2.

Maximum stress in concrete

$$= \frac{t}{m} \times \frac{n}{d-n} = \frac{1263 \times 18.69}{13 \times 43.31}$$

= 42 kg/cm^2.

Safe

Maximum shear force

= 4528 x 6 + 345.6 x $\frac{5}{2}$

= 27,168 + 864 = 28,032 kg.

Shear stress = $\frac{28,032}{0.87 x 62 x 30}$

= 17.28 kg/cm^2.

Shear force corresponding to shear intensity of 7 kg/cm.^{2}

= 7 x 0.87 x 62 x 30 = 11,327 kg.

Shear at 2m. from support

= 6 x 4528- ½ x 4 x 4528 x 2 + $\frac{345.6 x 5}{2}$ -345.6 x 2

= 9056 + 172.8 = 9228.8 kg.

No shear reinforcement is required beyond this point. 4 bars of 25 mm. ϕ, two at a time, and 2 bars of 32 mm. ϕ are bent at intervals of 0.5 m.

Shear taken by two bars of 32 mm. ϕ

= 2 x 1250 x 8.04 x 0.707

= 14,200 kg

Net shear = 28,032 - 14,200

=13,832 kg.

Providing 12 mm. ϕ 2-legged stirrups,

pitch = $\frac{2 x 1.13 x 1250 x 0.87 x 62}{13,832}$

= 108 cm.

12 mm. ϕ 2-legged stirrups at 8 cm. centres are provided.

Shear at 0.5 m. from support

= 28,032 - $\frac{0.5}{2}$ x w- $\frac{345.6}{2}$

= 28,032 - 1132 - 172.8

= 26,727.2 kg.

At this section two bars of 25 mm. ϕ bent up are effective,

Shear taken by 2 bars 25 mm. ϕ

= 2 x 1250 x 4.91 x 0.707

= 8700 kg.

Net shear for which stirrups are required

= 26,727.2 - 8700 = 18,027.2 kg.

Providing 12 mm. ϕ 2-legged stirrups at 8 cm. centres.

Shear at 1.5 m. from support

$$= 28{,}032 - \frac{3wx1.5}{2} - \frac{345.6x3}{2}$$

= 17,326 kg.

Provide 12 mm. ϕ 2-legged stirrups at 8 cm. centres upto 2 m. from support. In the remaining portion 12 mm. ϕ stirrups at 50 cm. centres are provided.

Design of Beam B_2. The side wall will serve the purpose of beam. It is assumed that weight of roof is taken by long walls.

Self weight of wall per metre run = 2112 kg.

Weight of water and base slab will be half of that coming on beam *B*1.

Maximum *B.M* . at centre

$$= 0.8\left[\frac{2112\times 5^2}{8} + 3\times 4528\times 2.5 - 2\times 4528 \times ½\times 2\left(\frac{2}{5}+0.5\right) - 2\times 4528\times 0.5\times 0.25\right]$$

$$= 0.8\left[\frac{2112\times 5^2}{8} + 3\times 4528\times 2.5 - 2 \times 4528\times \frac{7}{6} - \frac{4528}{4}\right]$$

= 23,090.4 kg. m.

Overall depth = 422 cm.

Assume effective depth = 416 cm.

Area of steel required = $\frac{23{,}090.4x100}{0.86x412x1250}$ = 4.9 cm^2

B.M at center of *AB*

$$= 18{,}0486 - \frac{14{,}475.1}{2}$$

= 10, 811 kg. m.

B.M at center of *BC* = 18,048.6 - 14,475.1

= 3573.5 kg. m

Taking moments about *B*,

R_A x 4 + 14,475.1- 3387 x 4 x 2- ½ x 4 x 2 x 4528 x 2 = 0.

R_A = 6774 + 9056-3350

= 12,480 kg.

R_D = 12,480 kg.

R_B = R_C = ½ [3387 x 12 + 3 x 1/2 x 4 x 2 x 4528 -2 x 12,480]

= 20, 322 + 27,168 - 12,480

= 35,010 kg.

S.F. diagram is shown in Fig. 32

Maximum *B.M* = 14,475.1 kg. m.

Overall depth of wall = 422 cm.

Assume effective depth = 416 cm.

Area of steel required = $\frac{14,475.1x100}{0.84x412x1000}$

= 4.16cm^2

Provide 3 bars of 16mm.∅.

Area of steel required at center of *AB*

$$=\frac{10.811x100}{0.86x416x1250} = 2.5 \text{ cm}^2$$

Provide 3 bars of 12mm.∅.

Area of steel required at center of *BC*

$$=\frac{3573.5\times10010.811\times100}{0.86\times416\times1250} = 0.8\text{cm}^2$$

2 bars of 16mm. ∅are provided.

Shear

Maximum shear force = 19,180 kg.

Shear stress = $\frac{19,180}{22\times0.87\times416}$

= 2.1 kg/cm^2. Safe

Provide 8mm. ∅stirrups at 30cm. centers

Design of columns

Load on column *A* when tank full = Reactions from beams B_2 and B_3

= 18,864 + 12,480 = 31.344 kg

Provide 3 bars of 20mm. ∅.

Shear force = 3 x 4528 + 2112 x $\frac{5}{2}$

=13,584 + 5280 =18,864 kg.

The section will be safe as depth provided is quite considerable.

Provide 8 mm. ∅ 2- legged stirrups at 30cm. centres.

Design of beam B_2. The side walls will serve as girders.

Load of roof and wall = 3387 kg/m.

Load from bottom slab will be triangular as shown in Figm 17m23 (a).

The beam is symmetrical and symmetrically loaded. Moments are found by moment distribution by cutting the beam at center of span *BC* and taking k = ½.

Due to triangular loading fixed end moments are

$$\frac{5wl}{48} = \frac{5}{48} x 2wx \frac{4}{2} x4$$

$$= \frac{5}{3} w = \frac{5}{3} x4528$$

= 7546.6 kg .m.

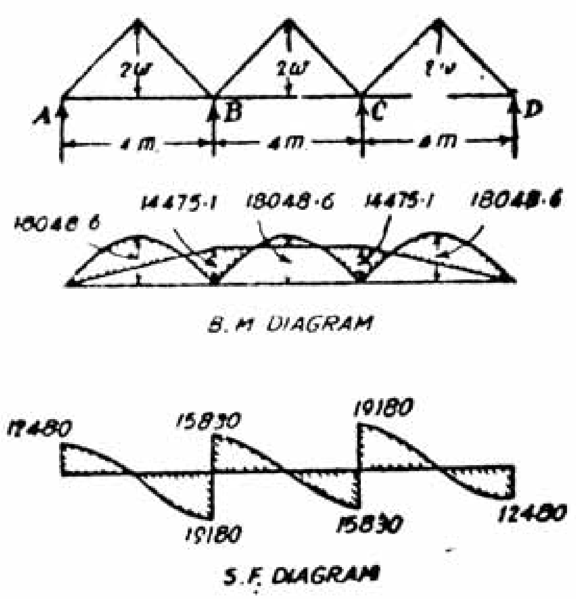


Fig 23

Due to uniformly distributed load fixed end moments are

$$\frac{wl^2}{12} = \frac{3387x4^2}{12} = 4516 \text{ kg. m.}$$

Total *F.E.M.* 7546.6 + 4516 =12,062.6 kg. m.

	A		3/5 B 2/5
F.E.M	-12, 062.6	+12, 062.6	-12, 062.6
Relese	+12, 062.6		
carry over		+6031.4	
Balance		+18,093.1	-12,062.6
		-3618.8	-2412.5
Final		+14,475.1	-14,475.1

Load on column *B* = 35.010 + 28,032

= 63,042 kg.

Total weight of water

= 12 x 5 x 4 x 1000 = 240,000 kg.

(For calculation of weight of water center line dimensions are taken as while calculating load on beams centre line dimensions were taken).

Columns *B,C,F* and *G* will take double the load of water taken by *A, E* , *D* and *H*.

Weight of water on column *A*

$$= \frac{240{,}000}{12} = 20{,}000kg.$$

Weight of water taken by column B

= 40,000 kg.

Load on column *B* when tank empty

= 31,344 - 20,000

= 11,344 kg.

Load on column B when tank empty

= 63,042-40,000

= 23,042 kg

Wind Forces

Intensity of wind pressure is assumed as 150 kg/m.^{2}

Wind force acting on water tank

= 4.37 x 12.22 x 150

= 8003 kg. acting at 14.185 from base

Wind Force on Staging

On columns = 4 x 0.3 x12 x150

= 2160 kg

On bracing = 3 x 0.3 x 12 x 150

= 1620 kg.

Total wind force on staging= 2160 + 1620 = 3780 kg. acting at 6m. from base.

Moment at base = 8003 x 14.185 + 3780 x 6

= 113,600 + 22,680

= 136.280 kg. m.

Σr^2 = 8 x 2.5 = 50

Force in column $= \frac{Mr}{\sum r^2} = \frac{136{,}280x2.5}{50}$

= 6814 kg. m.

There will be downward force of 6814 kg. in each of leeward columns and upward force on 6814 kg. in each of wind ward columns.

Self weight of column

= 0.3 x 0.4 x 12 x 2400 = 3456 kg

Maximum force in column

= 63,042 + 6814 + 3456

= 73,312 kg.

Maximum force in column

= 11,344 - 6814 = 5530 kg.

No upward load is produced in column when tank empty.

Design of column

Section of column is designed for dead load and checked for wind stresses.

Minimum dead load

= 63,042 + 3456

= 66,498 kg.

Section of 30 cm. x 40 cm. is provided.

66,498 = (30 x 40 - Ac) x 50 + Ac x 1300 Ac.

$$\therefore Ac = \frac{6498}{1250} = 5.20 \text{ cm}^2$$

4bars of 20 mm.ϕ are provide.

Area of steel provide

= 12.57 cm^2

Minimum diameter of ties

$$= \frac{20}{4} = 5.00 \text{ mm}$$

Provide 6 mm. ϕ ties at 30 cm. centres.

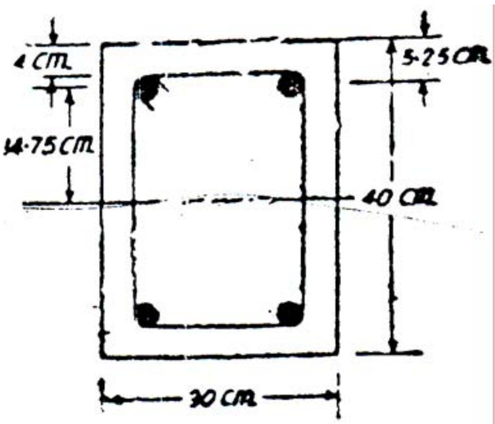


Fig. 24

Check for Stresses with Wind Blowing

Total horizontal force

= 8003 + 3780 = 11, 783 kg.

Force taken by each column

$$= \frac{11{,}783}{8} = 1473kg.$$

Maximum $B.M$ = 1473 x 1.5

= 2210 kg.m.

Maximum load = 73,312 kg.

$$\text{Eccentricity} = \frac{2210x100}{73.312} = 3.014 \text{ cm.}$$

As the eccentricity is very small no tension is induced.

Equivalent area of section

= 30 x 40 + 12 x 4 x 3.142

$= 1200 + 150.84 = 1350.84$ cm^2.

Equivalent moment of inertia

$$= \frac{1}{12} \text{ x } 30 \text{ x } 40^3 + 12 \text{ x } 4 \text{ x } 3.142 \text{ x } 14.75^2$$

$$= 160{,}000 + 32.790$$

$$= 192{,}790 \text{ cm}^4$$

Direct stress $= \dfrac{73342}{1350.84} = 54.28$ kg/cm^2.

Bending stress $= \dfrac{2210x100x15}{192{,}790} = 17.18$ kg/cm^2.

As the effect of wind is taken into consideration allowable stresses can be increased by $33\frac{1}{3}$%.

Allowable direct stress

$$= \frac{4}{3} \times 50 = 66.67 \text{ kg/cm}^2$$

Allowable bending stress

$$= \frac{4}{3} \times 70 = 93.33$$

$$\frac{54.28}{66.67} + \frac{17.18}{93.33} = 08139 + 0.1814$$

$$= 0.998 < 1 \text{ Safe.}$$

Design of Braces

Maximum B.M in the brase

= 2 x 2210 = 4420 kg. m.

= 442,000 kg. cm.

30 cm. x 40 cm. brace is provided.

Effective depth = 40 - 4 = 36 cm.

Moment of resistance of section

= Qbd^2 = 12.1 x 30 x 36^2

= 470,400 kg. cm.

Actual B M. is 442,000 kg. cm.

Area of steel required

$$= \frac{442{,}000}{0.87x36x1400} = 10.18 \text{ cm.}^2$$

Provide 4 bars of 20 mm. ϕ both at top and bottom as the wind may below from either side.

Maximum shear force

$$= \frac{2x1210}{5} = 884 \text{ kg.}$$

Shear force is very small. Sher stress is negligible.

Provide 6 mm ϕ stirrups at 30 cm. centres.

Design of foundation . For foundation concrete mix of M 150 is used.

Maximum load = 66,498 kg.
Assume self load = 6, 600 kg.
Total load = 73, 098 kg.
Area of footing required

$$= \frac{73{,}098}{15{,}000} = 4.87 m.^2$$

Provide 2.2 m. x 2.3 m. footing

Net upward pressure on footing = $\frac{66{,}498}{2.2x2.3}$ =13,140 kg/m^2

Maximum $B.M$ = 2.2 x 0.95 x $\frac{13{,}140x0.95}{2}$

= 13,040 kg. m.

Effective depth required

$$= \sqrt{\frac{13{,}040 \times 100}{8.7 \times 40}} = 61.22 \text{ cm.}$$

$$= \frac{66,498}{2 \cdot 2 \times 2 \cdot 3} = 13,140 \text{ kg/m}^2.$$

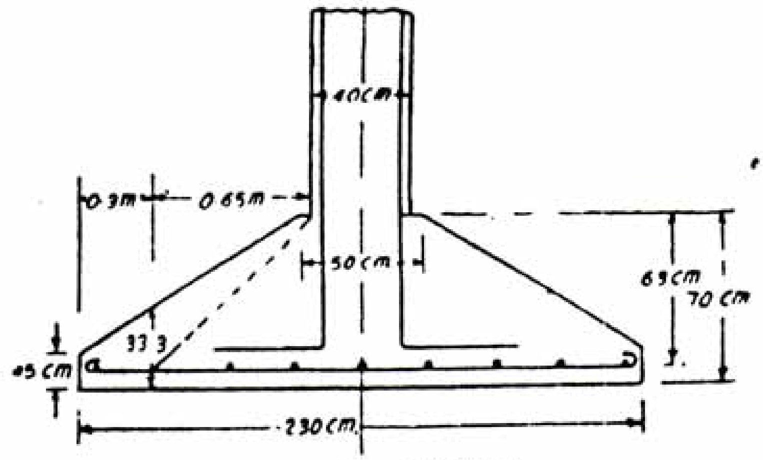


Fig. 17·25 (a)

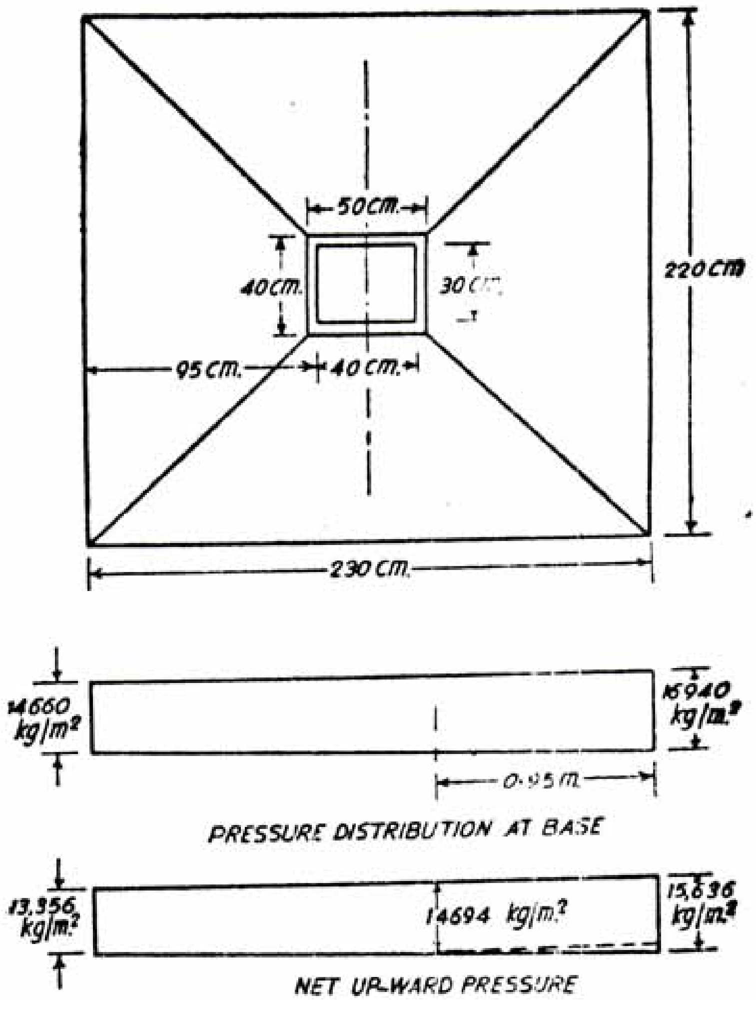


Fig. 17.24 (b)(c)

Depth required for punching

Punching force = 66,498 - 0.3 x 0.4 x 13,140

= 66,498 - 1,577 = 64,921 kg.

Effective depth required for punching

$$= \frac{64{,}921}{2\times(30+40)\times 10} = 46{,}37 \text{ cm.}$$

Provide overall deoth of 70cm.with effective depth of 63 cm.

Area of steel required for punching

$$= \frac{13{,}040\times 100}{0.87\times 6.3\times 1400} = 17 \text{ cm}^2$$

Provide 9 bars of 16 mm.φ in both directions.

Check for diagonal shear

Maximum shear = 0.3 x 2.2x13,140

= 8672.4 kg.

$$\text{Shear stress} = \frac{8672.4}{0.87x26.3xx160} = 2.37/\text{cm}^2$$

Safe.

Check for stresses due to wind load

Maximum vertical load = 73,312 kg.

Self load = 6600kg.

Total load = 79,912 kg

B.M = 2210 kg. m.

Pressure distribution on base is given by

$$p = \frac{79{,}912}{2.2\times 2.3} \pm \frac{2210}{2.2\times 2.3^2/6}$$

= 15,800± 1140

P_{max} = 16,940 kg/m^2

P_{min} = 14,660 kg/m$^{2.}$

Allowable pressure on ground considering wind effects

$$= \frac{5}{4} \text{ x } 15{,}000 = 18{,}750 \text{ kg/m}^2.$$

Pressure at base due to self weight of footing

$$= \frac{6600}{2.2\times 2.3} = 1304 \text{ kg/m}^2.$$

Net pressure on footing will very from 16,940-1304

= 15,636 kg/m$^{2.}$ to 14,660 - 1304

= 13,356 kg/m^2

B.M at the edge of column

$$= \left[14{,}694 \times \frac{0.95 \times 0.95}{2} + \frac{(15{,}636 - 14{,}694)}{2} \times 0.95 \right] \text{x } 2.2$$

= [6630 + 283.3] x 2.2

= 15,210 kg cm.

As the effect of wind is taken into consideration, stresses can be increased by $33\frac{1}{3}\%$

Effective depth required

$$= \sqrt{\frac{15{,}210 \times 100}{\frac{4}{3} \times 8.7 \times 40}}$$

= 54.1

Effective depth provided is 63 cm.

Hence O.K.

Area of steel required

$$= \frac{15{,}210 \times 100}{0.87 \times 63 \times 1400 \times \frac{4}{3}} = 15.2\text{cm}^2$$

Area of steel provide is 17cm^2. Safe.

REFERENCES

ARTHUR H. NILSON

"DESIGN OF CONCRETE STRUCTURES" , 12th Edition 1997