

**ANALYSIS & DESIGN OF OVERHEAD SERVICE RESERVOIR  
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Haryana, India.**Abstract**

In the construction of concrete structure for the storage of water & other liquid the imperviousness of concrete is most essential. The permeability of any uniform and thoroughly compacted concrete is given, mix proportion is mainly depend on the water cement ratio. The increase in water cement ratio results in increase the permeability. The decrease in water cement ratio may cause compaction difficulties and prove to be harmful also. For a given mix made with particular material there is a lower limit to the water cement ratio which can be used economically on any job. It is essential to select a richness of mix compatible with available aggregates whose particle shape and grading have an important bearing on workability, which must be suited to the means of compaction selected. Efficient compaction preferably by vibration is essential. It is desirable to specify cement content sufficiently high to ensure that through compaction is obtainable while maintaining a sufficient low water cement ratio. The quantity of cement should not be less than 320 kg/m<sup>3</sup> of concrete. It should also be less than 530 kg/m<sup>3</sup> of concrete to keep the shrinkage low. In thicker section, where a reduction in cement content might be desirable to restrict the temperature rise due to cement hydration, lower cement content is usually permissible. It is usual to use rich mix like M 30 grade in most of the water tanks. Design of liquid retaining structure has to be based on the avoidance of cracking in the concrete having regarded to its tensile strength. It has to be insured in its design that concrete does not crack on its water face. Cracking may also result from the restraint to shrinkage, free expansion & contraction of concrete due to temperature

and shrinkage & swelling. Due to moisture effects. Correct placing of reinforcement, use of small sized bars and used of deformed bars lead to a diffused distribution of cracks. The risk of cracking due to over all temperature and shrinkage effects may be minimized by limiting the changes in moisture content and temperature to which the structure as a whole is subjected. Cracks can be prevented by avoiding the use of thick timber shuttering which prevent the escape of heat of hydration from the concrete mass. The risk of cracking can be minimized by reducing the restraints on the free expansion or contraction of the structure. For long wall or slabs founded at or below the ground level, restraints can be minimized by founding the structure on a flay layer of concrete with interposition of sliding layer of some material to break the bond and facilitate movement.

However, it should be recognized that common and more serious causes of leakage in practice, other than cracking, are defects such as segregation and honey combing and in particular all joints are potential source of leakage.

**Sub-Area:** Structural Mechanics

**Broad Area:** Civil Engineering.

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### **Introduction:**

A water tank is used to store water to tide over the daily requirements. In general tanks can be classified under three heads : (i) tanks resting on ground (ii) elevated tanks supported on staging, and (iii) underground tanks. From the shape print of view, water tanks may be of several types, such as (i) circular tanks (ii) rectangular tanks (iii) spherical tanks (iv) Intze tanks and (v) circular tanks with conical bottoms.

In the construction of concrete structure for the storage of water & other liquid the imperviousness of concrete is most essential .The permeability of any uniform and thoroughly compacted concrete is given mix proportion is mainly depend on the water cement ratio. The increase in water cement ratio results in increase the permeability. The decrease in water cement ratio may cause compaction difficulties and prove to be harmful also. For a given mix made with particular material there is a lower limit to the water cement ratio which can be used economically on any job. It is essential to select a richness of mix compatible with available aggregates whose particle shape and grading have an important bearing on workability, which must be suited to the means of compaction selected. Efficient compaction preferably by vibration is essential. It is desirable to specify cement content sufficiently high to ensure that through compaction is obtainable while maintaining a sufficient low water cement ratio.

The quantity of cement should not be less than  $330 \text{ kg/m}^3$  of concrete. It should also be less than  $530 \text{ kg/m}^3$  of concrete to keep the shrinkage low. In thicker section, where a reduction in cement content might be desirable to restrict the temperature rise due to cement hydration ,lower cement content is usually permissible. It is usual to use rich mix like M 30 grade in most of the water tanks.

Design of liquid retaining structure has to be based on the avoidance of cracking in the concrete having regarded to its tensile strength. It has to be insured in its design that concrete does not crack on its water face. Cracking may also result from the restraint to shrinkage, free expansion & contraction of concrete due to temperature and shrinkage

& swelling. Due to moisture effects. Correct placing of reinforcement, use of small sized bars and used of deformed bars lead to a diffused distribution of cracks. The risk of cracking due to over all temperature and shrinkage effects may be minimized by limiting the changes in moisture content and temperature to which the structure as a whole is subjected. Cracks can be prevented by avoiding the use of thick timber shuttering which prevent the escape of heat of hydration from the concrete mass. The risk of cracking can be minimized by reducing the restraints on the free expansion or contraction of the structure. For long wall or slabs founded at or below the ground level, restraints can be minimized by founding the structure on a flay layer of concrete with interposition of sliding layer of some material to break the bond and facilitate movement.

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**General Design Requirements According to Indian Standard Code of Practice (IS: 3370-part II, 1965)**

**Permissible Stresses in Concrete**

**a. For resistance to cracking:**

Indian standard code IS: 456-1978 does not specify the permissible stresses in concrete for its resistance to cracking. However, its earlier version (IS: 456-1964) includes the permissible stresses in direct tension, bending tension and shear. These values are given in table ..... permissible tensile stresses due to bending apply to the face of member in contact with the liquid. In members with thickness less than 225mm and in contact with the liquid on one side, these permissible stresses in bending apply also to the face remote from the liquid.

**TABLE- 1**

**PERMISSIBLE CONCRETE STRESSES IN CALCULATIONS RELATING TO RESISTANCE TO CRACKING.**

Permissible Stresses			
Grade of concrete	Direct Tension $Q^{st}$	Tension due to bending $Q_{cbt}$	Shear = $Q_{bjd}$
	N/mm <sup>2</sup>	N/mm <sup>2</sup>	N/mm <sup>2</sup>
M15	1.1	1.5	1.5
M20	1.2	1.7	1.7
M25	1.3	1.8	1.9
M30	1.5	2.0	2.2
M35	1.6	2.2	2.5
M40	1.7	2.4	2.7

- b. **For strength calculations:** in strength calculations the usual permissible stresses in accordance with IS 456 - 1978 are used. Where the calculated shear stress in concrete above exceeds the permissible value, reinforcement acting in conjunction with diagonal compression in concrete shall be provided to take the whole of the shear.

### PERMISSIBLE STRESSES IN REINFORCEMENT

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

- c. **For strength calculations :** Though the Indian Standard Code IS : 456 had its third revision in 1978, the corresponding codes IS : 3370 n (part I,II,III,IV) for concrete structures for the storage of liquids have not been revised since 1965. The main code on concrete – IS: 456 are SI units. However, the fourth reprint (May 1982) of IS: 3370 (part II) -1965 incorporates the amendment regarding the permissible stresses in steel reinforcement. The revised values of permissible stresses are given in table ..... Converted in to SI units, using the approximation  $10 \text{ kg/cm}^2 = 1 \text{ N/mm}^2$ .

**Note:** Stress limitations for liquid retaining faces shall also apply to the following:

- (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.

**Table: 2**  
**Permissible Stresses in Steel Reinforcement for Strength Calculation**

Types of stress in steel reinforcement	Plain round mild steel bars conforming to grade 1 of IS : 482 (part I) 1996	High yield Strength: deformed bars (HYSD) conforming to IS: 1789- 1966 or IS 1139- 1966.
1. Tensile stress in members under direct tens( $Q_s$ )	115	150
2. Tensile stress in members due to bending ( $Q_{st}$ )		
(a) On liquid retaining face of members.	115	150
(b) on face away from liquid for members less than 225 mm	115	150
(c) on face away from liquid for 225mm or more in thickness.	125	190
3. tensile stress in shear reinforcement( $Q_{sv}$ )		
(a) for members less than 225mm thickness	115	150
(b) for members less than 225 or more in thickness		
4. compressive stress in columns subjected to direct load( $Q_{sc}$ )	125	175
	125	175

**STRESSES DUE TO DRYING SHRINKAGE OR TEMPERATURE CHANGE:**

(1) Stresses due to drying or temperature change may be ignored provided that:

(a) The permissible stresses specified in para (2) and (3), for concrete and steel respectively are not exceeded.

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

(c) The recommendations as regards the provision of joint and for suitable sliding layer (see .....19.3) 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.

(2) Shrinkage stresses may, however, be required to be calculated in special case, when a shrinkage coefficient of  $300 \times 10^{-6}$  may be assumed.

(3) When shrinkage stresses are allowed, the permissible stresses, tensile stresses in concrete (direct and bending) as given table.....may be increased by  $33^{1/3}$  percent.

(4) Where reservoir is protected with an internal impermeable lining, consideration should be given to the possibility of concrete eventually drying out. Unless it is established on the basis of tests or experience that the lining has adequate crack bridging properties, allowance for the increased effect of drying shrinkage should be made in the design.

## STEEL REINFORCEMENT

### a. Minimum reinforcement

(i) 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 up to 100 mm thickness. For sections of thickness greater than 100 mm and less than 450 mm the minimum reinforcement in each of two directions shall be linearly reduced from 0.3 percent for 100 mm thick section to 0.2 percent for 450 mm, minimum reinforcement in each of two directions shall be kept at 0.2 percent. In concrete sections of thickness 225 mm or greater, two layers of reinforcing bars shall be placed one near each face of the section to make up the minimum reinforcement specified above.

(ii) In special circumstances, floor slabs resting directly on the ground may be constructed with percentage of reinforcement less than that specified above in no case the percentage of reinforcement in any member be less than 0.15 % of the concrete section.

### b. Minimum cover to reinforcement :

- i. for liquid faces of parts of members either in contact with the liquid or enclosing the space above the liquid (such as inner face of slab); the minimum cover to all reinforcement should be 2.5 mm or the diameter of the main bar, whichever is greater. In the presence of sea water and 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.
- ii. For faces away from the liquid and for parts of the structure neither in contact with the liquid on any face for enclosing the face above the liquid, the cover should be the same as provided for other reinforced concrete section.

## DESIGN OF 75,000 LITRES CAPACITY OHSR F.S.L 22.00 M

### Tank Dimensions:

Keep c/c of columns = 5.41m

Rise of top dome = 1.00m

Rise of bottom dome = 1.15m

Radius of bottom dome =  $(2.505^2 + 1.15^2)/2 \times 1.15 = 3.303\text{m}$

Vol. occupied by bottom dome =  $3.142 \times 1.15^2 (3.303 - 1.15/3) = 12.13\text{m}^3$

If height of water is 'h' m. then  $3.141 \times 5.59^2 \times h/4 = 75 + 12.13$ ;  $h = 3.55\text{m}$

Provide 3.55m height of water and 0.6m free board so the total height of tank 4.15m

TOP DOME :( 7.5 Cm THICK )

Radius of top dome =  $( 2.805^2 + 1.0^2 )/2 \times 1.0=4.434\text{m}$

$\sin \theta = 2.805/4.434 = 0.633$  ;  $\cos \theta = 3.434/4.434=0.775$

Dead load =  $262.5 \text{ kg/m}^2$

Thrust at edges =  $262.5 \times 4.434 / ( 1 + 0.775 ) = 656.0 \text{ kg/m}$

Compressive stress =  $656.0 / 7.5 \times 100 = 8.75 \text{ kg/cm}^2$

Minimum steel = .24% of concrete area =  $.24 \times 7.5 \times 100 / 100 = 1.80 \text{ cm}^2$

So provide 8mm dia. Bars 225mm c/c both ways in form of mesh.

Provide 2no. 10mm dia. Circular rings around the opening ( manhole ) with one leg projection in to dome up to length of 450mm. provide 6mm dia. @ 150mm c/c dome connectors with one leg of length 600mm projection into springing of dome.

TOP RING BEAM :( 100 X 150 MM )

Horizontal force =  $656 \times 0.775 = 509 \text{ kg/m}$

Hoop tension =  $509 \times 5.70 / 2 = 1451 \text{ kg}$

Area of steel bars =  $1451 \times 56 / 1500 = .97 \text{ cm}^2$

Provide 4 bars of 8mm dia. , 2 at the top and bottom each and 6mm Dia. Stirrups 150c/c with one leg.

Projecting into top dome equivalent area of section =  $15 \times 10 + 12 \times 4 \times .5 = 174 \text{ cm}^2$

Tensile stress =  $1451 / 174 = 8.34 \text{ kg/cm}^2 < 12 \text{ kg/cm}^2$  ( safe )

Dead load steps =  $0.91 \times 0.2635 \times 0.1939 \times 2500 \times 13 / 2 = 756 \text{ kg}$

W.m.r.R. =  $1000 \times 5.59 \times 3.55 / 2 = 9923$

Refer page 38 Table 12 of I.S. 3370 Part 1V

Depth	Coeff	Hoop force	Steel required	Steel provided
0.0H	0.002	20	0.02 Cm <sup>2</sup>	8mmØ rings @ 252 mm C/con each
0.1H	0.100	993	0.67 Cm <sup>2</sup>	face in 1260mm ( 10no rings = 5+5 )
0.2H	0.198	1965	1.31 Cm <sup>2</sup>	
0.3H	0.299	2967	1.98 Cm <sup>2</sup>	4H 0.403 3999 2.67 Cm <sup>2</sup> 8mmØ rings @ 280mm C/c on each
0.5H	0.521	5170	3.45 Cm <sup>2</sup>	face in 840 mm ( 6no rings = 3+3 )
0.6H	0.650	6450	4.30 Cm <sup>2</sup>	
0.7H	0.764	7581	5.06 Cm <sup>2</sup>	8mmØ rings @ 190 mm C/c on each
0.8H	0.776	7701	5.14 Cm <sup>2</sup>	face up to 1900 mm ( 20 rings = 10+10 )
0.9H	0.536	5319	3.55 Cm <sup>2</sup>	

Minimum steel required = 0.24% = 2.64 Cm<sup>2</sup>

Referring table 10 & 13.I.S.3370 Part 1V fully fixed wall.

Bending moment = 0.0079x 1.000x3.555<sup>3</sup>=353.5kg-m

Thickness of wall required = 353.5x100/17x1000=11.16cm.

So depth provided 12.0cm is O.K.

Area of steel for B.M. = 353.5x100/1500x0.874x8.5=3.18 Cm<sup>2</sup>

Provide 8mm Ø mm c/c on inner face vertically throughout and additional bars 10mm Ø 360mm C/c on inner face up to 1200 mm height & 8mmØ360mm c/c on outer face.

Equivalent area at 0.8 H =100x11.6++12x0.5/0.095=1223.0 Cm<sup>2</sup>

Tensile stress = 6792/10x100=6.80<50x2/5=20kg/ Cm<sup>2</sup>

Bottom dime: ( 100 mm thick )

Self wt = 2x3.141x3.303x1.15x10.0x25=5966kg.

Wt.Of water = 75000kg.

Loan per Sq.m= 80966/2x3.141x3.303x1.115=3393kg/m<sup>2</sup>

Sin Ø=2.505/3.303=0.76 Cos Ø=(3.303-1.15)/3.303=0.65

Meridional Thrust = 3393x 3.303/ ( 1+0.65)=6792kg/m

Meridional Stress = 6792/10x100=6.80<50x2/5=20kg/ m<sup>2</sup>

Load from vertical wall =3.141x5.70x0.11x4.15x2500=20440

Wt. of bottom dome = 5966kg.

Self weight of beam = 3.142x5.41x0.40x0.65x2500=11049

**Gallary**:=3/4x3.142x6.71x0.9x0.10x2500=3558kg.

Wt. of Water = 75,000kg.

Parapet of top =3.141x5.71x0.1x2500x0.91=4082kg.

Live Load = 3/4x3.142x6.71x0.91x150=2159kg.

Total Load = 129566kg.say 130000kg.

Load per Meter = 130000/5.41x3.142=7648kg.

Maximum support moment = 0.137x7648x2.705<sup>2</sup>x3.141/2=12045kg-m

Maximum mid span moment = 0.07x7648x2.705<sup>2</sup>x3.141/2=6155kg-m

Maximum twisting moment = 0.021x7648x2.705<sup>2</sup>x3.141/2=1847kg-m

Eff. Depth required = 14897x100=60.72cm

10.1x40

Provided D = 65 cm de = 61 cm

MT =1847 ( 1+650/400 )/1.7=2892 kg -m

Max design moment at support = 12045+2852 kg -m

Max design moment at support = 12045+2852 kg -m

Max design moment at mid span =6155+2852= 9007 kg -m

Area of steel support = 14897x100/1500x0.9x61.0= 18.009cm<sup>2</sup>

Area of steel support = 9007x100/1900x0.9x61.0= 8.94cm<sup>2</sup>

Provide6,20mmØat top = 18.85cm<sup>2</sup> curtail 2,20mmØat 8.30mfrom column edge.



Provide  $4, 16\text{mm}\varnothing + 1.10\text{mm}\varnothing$  at bottom throughout =  $8.82\text{cm}^2$   
 Side face reinforcement =  $0.1 \times 40 \times 65 = 2.60\text{cm}^2$  i.e.  $1.30\text{cm}^2$ ,  $10\text{mm}\varnothing$  on each side.  
 Ast at supports =  $18.85 + 3.142 + 8.82 = 30.81\text{ cm}^2 > 181.09 + 7.97 = 26.06\text{ cm}^2$   
 Ast at mid span =  $12.56 + 3.142 + 8.82 = 24.53\text{ cm}^2 > 8.64 + 7.97 = 16.61\text{ cm}^2$   
 S.F. at edge of column  $7648 \times 3.142 \times 5.41 / 8 - 7648 \times 0.2 = 14721\text{kg}$ .  
 S.F. at point of max torsion =  $2.705(45^0 - 19.25^0) 3.141 \times 7648 / 180 = 9300$   
 S.F. at due to torsion =  $1.6 \times 1847 / 0.4 = 7388\text{kg}$ .  
 Total S.F. Point of maximum torsion =  $9300 + 7388 = 16688\text{kg} > 14721\text{kg}$   
 $A_s / b d = 100 \times 8.82 / 40 \times 61.0 = 0.36$   
 Allowable shear stress  $t_c = 2.45\text{kg./cm}^2$   
 $V - C_c b d = 16688 - 2.45 \times 40 \times 61 = 10710$   
 Spacing of  $10\text{mm}\varnothing$  2 legged stirrups =  $2 \times 1.13 \times 1500 \times 61 / 10710 = 14.350\text{ cm C/c}$   
 So provide 2 legged.  $10\text{mm}\varnothing$  stirrups @  $145\text{mm c/c}$  throughout  
**Gallery :** Self Wt. of  $10\text{cm}$  slab =  $25\text{-kg/cm}^2$   
 L.L. =  $300.0\text{ kg/m}^2$  Total Load =  $550\text{kg/ cm}^2$   
 B.M. =  $550 \times 0.9^2 / 2 = 223\text{kg/m}$   
 $D_e = 223 \times 100 / 9.1 \times 100 = 4.95\text{cm}$   
 Provide,  $D = 100\text{mm}(d_e = 65\text{mm})$   
 $A_t = 223.0 \times 100 / 1900 \times 0.892 \times 6.5 = 2.03\text{ cm}^2$   
 Minimum steel required =  $0.24\% = 2.4\text{ cm}^2$   
 Spacing of  $8\text{mm}\varnothing$  bars =  $0.5 \times 100 / 2.4 = 20/0\text{cm c/c}$   
 Provide  $8\text{ mm } \varnothing$  radial bars @  $200\text{ c/c}$  and 5 Nos. rings  $8\text{ mm } \varnothing$  as shown in drawing.

### Parapet Wall

Provide  $100\text{ mm}$  thick parapet wall reinforce with  $8\text{ mm } \varnothing$  @  $200\text{ c/c}$  both ways as shown.

Max vertical load in extreme columns =  $112.83 / 5.41 = 20856\text{kg}$

Direct stress =  $69856 / 1387.31 = 50.36\text{kg/cm}^2$

Seismic shear per column =  $1/4 \times 5140 = 1285\text{kg}$

B.M. in column =  $1767 \times 100 / 7057.8 = 25.04\text{kg/ cm}^2$

bending stress =  $1767 \times 100 / 7057.8 = 25.04\text{kg/ cm}^2$

### Check For

**stresses :**  $50.36 / 60 \times 1.333 + 25.04 / 85 \times 1.333 = 0.63 + 0.23 = 0.86 < 1.00$

### Tank Empty Case

Equivalent weight  $W = 151.50 - 75 = 76.50\text{T}$

Time Period =  $2 \times 3.142 \sqrt{(76.50 / 9.81 \times 662)} = 0.68\text{sec}$  sag =  $0.14$

Horizontal seismic coeff. =  $1.0 \times 1.5 \times 0.2 \times 0.14 = 0.042$

Horizontal seismic force =  $0.042 \times 76.50 = 3.22T$

Turning moment of wind above foundation top =  $3.22 \times 21.95 = 70.68 \text{ t-m}$

Max. vertical load in extreme column =  $70680/5.41 = 13065 \text{ kg}$

### Check For Stresses

Min, axial force in column =  $49000 - 75000/4 - 13065 = 17185$

Direct shear stress in column =  $17185/1387.31 = 12.39 \text{ kg/cm}$

Net tensile stress =  $25.04 - 12.39 = 12.65 \text{ kg/cm}^2 <^{3/4} \times 27 \times$

$4/4 = 27 \text{ kg/cm}^2$  (o.k safe)

Design of braces: Seismic shear in each bay =  $1285 \text{ kg}$

Wind shear in bottom bay =  $1742 \text{ kg}$

Wind shear in second bay =  $\frac{1}{4} (3153 + 3/4 \times 1423 + 2391 \times 14.55/16.55) = 1581 \text{ kg}$

Wind shear in third bay =  $\frac{1}{4} (3153 + 2/4 \times 1423 + 2391 \times 14.55/16.55) = 1378 \text{ kg}$

For 4<sup>th</sup> & 5<sup>th</sup> bay seismic shear will be dominating

B, Min 1<sup>st</sup> brace from bottom =  $(1742 \times 2.975/2 + 1581 \times 3.60/2) = 5437 \text{ kg-m}$

B, M in 2<sup>nd</sup> brace from bottom =  $(1581 \times 3.60/2 + 1378 \times 4.25/2) = 5775 \text{ kg-m}$

B, M in 3<sup>rd</sup> brace from bottom =  $(1378 \times 4.25/2 + 1285 \times 4.25/2) = 5659 \text{ kg-m}$

B, M in 3<sup>rd</sup> and 4<sup>th</sup> brace from bottom =  $1285 \times 4.25 = 5462 \text{ kg-m}$

Design moment =  $5775 \text{ kg-m}$

Torsional moment =  $5\% = 289 \text{ kg-m}$

B.M due to torsion =  $289 (1 + 45/20) 1.7 = 553 \text{ kg-m}$

Total B.M =  $5775 + 553 = 6328 \text{ kg-m}$

$A_{st} = 6328 \times 100 / 2300 \times 37 \times 1.333 = 5.58 \text{ cm}^2$

Provide 3, 16 dia at top and bottom =  $6.03 \text{ cm}^2$

Total shear force =  $2 \times 5775 / 3.42 + 1.6 \times 289 / 0.20 = 5690 \text{ kg}$

$100 A_{st} / b d = 100 \times 6.03 / 20 \times 37 = 0.82 C_c = 3.6 \text{ kg/cm}^2$

Shear force taken by concrete =  $3.6 \times 20 \times 37 = 2664 \text{ kg}$

Sacing of 8mm dia D legged stirrups =  $2 \times 0.5 \times 2300 \times 37 / (5690 - 2664) = 28.12 \text{ cm c/c}$

Provide 2 legged 8mm dia. Stirrups @ 27cm c/c throughout in all braces.

### Design Of Foundation

Load transferred through 4 column =  $49000 \times 4 = 196000 \text{ kg}$

Add 10% wt. of foundation =  $20000 \text{ kg}$

Total load =  $216000 \text{ kg}$

Area of foundation required =  $216 / 8.75 = 24.69 \text{ M}^2$

Wind pressure on tank portion and ring beam =

$(4.15 + 9 + 0.65) \times 5.81 \times 0.7 \times 136 = 3153 \text{ kg}$

Acting at  $19.10 + 5.70/2 = 21.95 \text{ m}$  above foundation top

Wind force on coloumns =  $16.55 \times 3 \times 0.40 \times 0.7 \times 136 = 1891 \text{ kg}$  at

$18.35/2 + 0.75 = 9.925 \text{ m}$  above beam

Wind load on braces= $4 \times 0.45 \times 5.81 \times 136 = 1423 \text{kg}$

Wind load on stairs= $500 \text{kg}$

Total wind force on columns and braces= $1891 + 1423 + 500 = 3814 \text{kg}$  acting

At 9.925m above F beam

Overturning moment about top of F

Beam= $3814 \times 9.925 + 3153 \times 21.95 = 107063 \text{kg-m}$

Max load in the remotest column= $107063 / 5.41 = 19790 \text{kg}$

**Tank Full:**

Max. total load= $49000 + 19790 = 68790 \text{kg}$

Direct stress= $68790 / 1387.31 = 49.59 \text{ kg/cm}^2$

B.M in column= $1742 \times 2.75 / 2 = 2396 \text{kg-m}$

Bending stress= $2396 \times 100 / 7057.8 = 33.95 \text{kg/cm}^2$

Check for stress= $49.59 / 60 \times 1.333 + 33.95 / 85 \times 1.333 = 0.63 + 0.30 = 0.93 < 1$  (ok)

Tank Empty: minimum axial load= $49000 - 19790 - 75000 / 4 = 10460 \text{kg}$

Direct stress= $10460 / 1387.31 = 7.54 \text{kg/cm}^2$

Net tensile stress= $33.95 - 7.54 = 26.41 \text{kg/cm}^2 < 3/4 \times 27 \times 1.33 = 27 \text{kg/cm}^2$  (ok)

**Analysis Due To Seismic Forces:**

Tank full: Wt. of main tank =  $130.0 \text{T}$

Wt. of staging  $W_2 = 4 \times 6.00 + 12.32028 = 64.32 \text{T}$

Equivalent wt. =  $w^1 + w_2 / 3 = 30.0 + 64.32 / 3 = 151.50$

$K = 4 \times 12EI / 13 = 4 \times 12 \times 2085 \times 10^6 \times 141156 \times 10 / (2.975^3 + 3.60^3 + 4.253 + 4.02533) = 662 \text{t/m}$

Time period =  $2 \times 3.142 (151.50 / 9.81 \times 662) = 662 \text{t/m}$

Time period =  $2 \times 3.142 (151.50 / 9.81 \times 662) = .96 \text{sec}$

$S_a / g = 0.113$

The tank is to be constructed in seismic zone 3

Horizontal seismic coeff. =  $1.0 \times 1.5 \times 0.2 \times 0.113 = .0339$

Horizontal seismic force =  $0.0339 \times 151.50 = 5.14 \text{t}$  acting at 21.95m above the F.beam

Turning moment about foundation top =  $5.14 \times 21.95 = 112.83 \text{t-m}$

Try annular raft outer diameter = 7.15m and inner diameter = 3.67m

$A = 3.142 / 4 (7.15^2 - 3.67^2) = 29.57 \text{m}^2$

$I = 3.142 / 64 (7.15^4 - 3.67^4) = 119.40 \text{m}^2$

$Z = 119.40 / 3.575 = 33.39 \text{m}^3$

**Check For Wind Forces:**

O. moment due to wind forces =  $107063 + 0.8 \times (3153 + 3814) = 112637 \text{kg-m}$

Bending pressure due to wind pressure =  $112.64 / 33.39 = 3.38 \text{ t/m}^2$

Direct pressure =  $7.31 + 3.38 = 10.69 \text{ t/m}^2 < 1.25 \times 8.750 \text{ t/m}^2$

When the tank is empty (  $216.0 - 75$  ) /  $29.57 - 3.38 = 4.77 - 3.38 = +ve$

Hence , no uplift is developed when the tank is empty O.K. (safe)

### Check For Seismic Forces:

O. moment due to seismic forces =  $112830+5140 \times 0.8=116942 \text{kg-m}$

Bending pressure due to wind pressure =  $116.95/33.39=3.51 \text{ t/m}^2$

Direct pressure =  $216.0/29.57=7.31 \text{ t/m}^2$

Total pressure =  $7.31+3.51= 10.82 \text{ t/m}^2 < 1.33 \times 8.750 \text{t/m}^2$

When the tank is empty  $(216.0-75)/29.57-3.51=4.77-3.51 = +ve$

Hence , no uplift is developed when the tank is empty O.K. (safe)

### Design of foundation ring beam:

Total load of coming on the girder =  $196.0/3.142 \times 5.41=11531 \text{kg}$ .

Maximum B.M. at support =  $0.137 \times 11531 \times 2.705^2 \times 3.142/2=18.160 \text{t-m}$

Maximum B.M. at mid span =  $0.07 \times 18.16/0.137=9.28 \text{t-m}$

Maximum tensional moment =  $0.021 \times 18.16/0.137=2.784 \text{t-m}$

Equivalent B.M. due to torsion =  $2784(1+800/400)/1.7=4913 \text{kg-m}$

Total - ve B,m. at support =  $18160+4913=23073$

Total - ve B,m. at mid span =  $9280+4913=14193 \text{kg-m}$

Effective depth required =  $18160 \times 100 \times 1000/9.1 \times 40=70.6 \text{cm}$

So provided depth = 80.0cm is O.k.

Ast at support =  $23073 \times 100 \times 1000/2300 \times 0.9 \times 74.0=15.07 \text{ cm}^2$

Ast at mid span =  $14193 \times 100 \times 1000/2300 \times 0.9 \times 74.0=9.27 \text{ cm}^2$

Provide 5,20 mm Ø at bottom Ast =  $15.70 \text{ cm}^2$  out of which curtail 2,20mm

Ø at 1.3 m from support

At top, provide 3,20mmØ =  $9.42 \text{cm}^2$  provide 1,16 mmØ on each face of ring beam

S.F. at face of column =  $11531 \times 3.14 \times 5.41/8-11531 \times 0.2=22195 \text{kg}$

S.F. at point of contra flecure =  $3.142 \times 5.41 \times 25.75^0 \times 11531/180 \times 2=14020 \text{kg-m}$

Torsional shear =  $1.6t/b = 11136 \text{kg}$  Design shear = 25156kg

$100 \text{ As}/bd = 100 \times 9.42/40 \times 74=2.40 \text{kg}/\text{cm}^2$

Shear taken by concrete =  $2.40 \times 40 \times 74=7104 \text{kg}$

Balance shear =  $V-Cc \text{ bd}= 25156-7104=18052 \text{kg-m}$

Spacing of 10mm Ø 2 legged strriups =  $2 \times 0.785 \times 300 \times 74/18052=14.80 \text{CmC}/c$

Provide 2 legged 10mm Ø shear stirrups @ 145 mm C/c throughout.

### Design of stair case:

Total height =  $22.0-3.0=19.0 \text{m}$

Provide 7 flights each of 2.714m height

Horizontal distance to be covered by each flight = 3.425m

Provide 14 raisers in each flight & 13 trends in each flight

Size of tread = 263.5mm, size of risers =  $2.714/14=0.1939$

Minimum reinforcement 0.24%=2.40cm

Provide 8mm Ø bars @ 200mm C/c both ways

Provide 3-8mmØ hoop rings near springing in 800mm length.

**(a) Bottom Ring Beam:**

Horizontal force from dome = 6792x0.65=4415kg/m

Hoop Tension = 4415x5.41/2=11943kg.

Area of Hoop steel = 11943/1500=7097cm<sup>2</sup>

**(b) DESIGN FOR VERTICAL LOAD:**

**Case I.** Load from top dome = 2x3.141x4.434x1.0x262.5=7312kg.  
=4072kg-m

**Case II.** Forces of downward ring beam load on slab

$$M_0 = w \times \frac{a^2}{4} \times \frac{(1+b^2)}{a^2-b^2} \left[ \frac{\text{Log}_e a/c + \frac{1}{2}c^2}{r^2} \right] \times \frac{1}{2a^2}$$

=6271kg-m

Net M<sub>0</sub>=20795-18662=2133kg-m

M<sub>0</sub> = 6271-4072=2199kg-m

**(c) Selection at the outer edge of the slab :-**

**Case I.** Forces of upward soil pressure on slab : M<sub>r</sub> =0

$$M_0 = -pr^2 + pb^2 \left( \log_e r + \frac{3}{4}(-\frac{1}{3} + a^2 + a^2) \right) - \left( \frac{a^2+r^2}{r^2} \cdot b^2 \log_e a/b \right)$$

=13901kg-m

**Case II.** Forces of downward ring beam load on slab:- r = a = 3.575<sub>m</sub>

$$M_0 = w \left( \frac{\log_e c + \frac{1}{2} + a_2}{4r} \cdot \frac{a_2 - b_2}{r_2} \right) \left[ \frac{\text{Log}_e a/c + \frac{1}{2}c_2}{2a_2} \right] \cdot c_2$$

=15780kg-m

Net M<sub>0</sub> = 15780-13901 = 1879kg-m

**(d) Selection at the outer edge of ring beam :-**

**Case I.** Forces of upward soil pressure on slab

$$M_0 = -\frac{pr^2}{16} + \frac{pb^2}{4} \left( \log_e r + \frac{3}{4}(-\frac{1}{3} + a^2 + a^2) \right) - \left( \frac{a^2+r^2}{r^2} \cdot \frac{b^2}{a^2-b^2} \log_e a/d \right)$$

$$M_r = -\frac{3pr^2}{16} + \frac{pb^2}{4} \left( \log_e r + \frac{3}{4}(1+a^2 + a^2) \right) - \left( \frac{a^2+r^2}{r^2} \cdot \frac{b^2}{a^2-b^2} \log_e a/d \right)$$

=3590 kg – m

**STAGING:** F.S.L = 22.0m

Provide 4no circular columns 40cm diameter.

Height up to top ring beam = 22.0-3.0-0.065+0.75=19.10m above

Foundation top

No of braces = 4, size of braces = 200x450mm size

Provide bay height 2.75m,3.15m,3.80m and 3.80m clear distance between braces.

Load per column tank = 130000/4=32.50T

Self weight for column =  $3.142/4 \times 0.4 \times 0.4 \times 19.10 \times 2500 = 6.00t$

Wt. of braces =  $4 \times 3.42 \times 0.20 \times 0.45 \times 2500 = 3078kg$ .

Wt. of stair case for each column =  $32.50 + 6.00 + 3.08 + 7.0 = 48.58 T$  say 49 T

Area of concrete required =  $49 \times 1000 / 60 = 817cm^2$

Area provided =  $3.142 \times 40 \times 40 / 4 = 1256.8cm^2$  is O.K.

Area of steel required =  $0.008 \times 817 = 6.54cm^2$

provide 7,12mm dia and 6mm dia ties at 190c/c throughout M25 conc.

Mix up to top through out .

Area of equivalent section =  $3.142/4(40 \times 40) + (16.5 \times 7.91) = 1387.31cm^2$

$I = 141156cm^4$

Section modulus =  $141156 / 20 = 7057.85 cm^3$

### ANALYSIS DUE TO WIND FORCES :

#### WIND PRESSURE:

total height of tank =  $22.55 + 0.6 + 0.9 = 24.05m$

Wind speed = 47m/sec

Assuming terrain category =2 and class B, reoffering I.S code 875(part-3)-1987

Height	K2	Pz(n/mm <sup>2</sup> )	Pz(kg/cm <sup>2</sup> )
10.0	0.98	1273	127.3
15.0	1.02	1379	137.9
20.0	1.05	1462	146.2
24.05	1.075	1532	153.2

#### Average Wind:

Pressure =  $127.3 \times 10 + 132.60 \times 5 + 142.05 \times 5 + 149.70 \times 4.05 / 24.05 = 135.3kg/cm^2$

say  $136kg/cm^2$  inclined length of waist slab =  $\sqrt{(2.714^2 + 3.825^2)} = 4.69m$

wt. of slab =  $4.69 \times 0.91 \times 0.16 \times 2500 = 1708kg$ .

live load 500kg, total load = 2964kg

B.M =  $2964 \times 3.83 / 10 = 1136kg-m$

$De = \sqrt{[1136 \times 100 / (91 \times 91)]} = 11.71cm$

Provide D = 16.0cm

Area of steel =  $1136 \times 100 / 2300 \times 0.90 \times 12.5 = 4.39cm^2$

Provide 6 nos. 10mm dia. And 8mm dia. @ 25cm c/c as temp steel.

#### DESIGN OF LANDING:

Load from flight = 2964kg

Self weight of landing = 700kg (say), total load = 3664kg

Approximate distance of C.G of landing from column =  $2/3 \times 0.9 = 0.6m$

B.M =  $3664 \times 0.60 = 2199kg-m$

Effective depth required =  $\sqrt{(2199 \times 100 / 9.1 \times 40)} = 24.58cm$

So provide over all depth  $D=32\text{cm}$  reduced to  $16\text{ cm}$  near outer edge

$$\text{Ast required} = 2199 \times 100 / 2300 \times 0.90 \times 28.5 = 3.73 \text{cm}^2$$

Provide 6,10mm dia. At top and 6,10mm dia. At bottom.

### DESIGN OF FOUNDATION SLAB BY PLATE LOAD THEORY:

$$\text{Intensity of pressure (p)} = 10.69 / 1.25 = 8552 \text{kg/Cm}^2 \text{ say } 8600 \text{ kg/ Cm}^2$$

$$\text{Outer radius of foundation slab (a)} = 3.575 \text{m}$$

$$\text{Inner radius of foundation slab (b)} = 1.835 \text{m}$$

$$\text{Central radius of foundation slab (c)} = 2.705 \text{m}$$

#### A. Section At The Inner Edge Of Slab:

$$r = 1.835 \text{m}$$

Case I : forces of upward soil pressure on slab;  $M_r=0$

$$\begin{aligned} M_o &= pr^2/16 + pb^2/4 [(\log_e r/a + 3/4(-1/3 + a^2/b^2 + a^2/r^2))] - [(a^2+r^2)/r^2, b^2/(a^2-b^2) \log_e a/b] \\ &= 24482 \text{ kg-m} \end{aligned}$$

Case II. Forces of downward ring beam load on slab ;  $M_r=0$

$$W = 8600 \times 29.57 = 254302 \text{ kg}$$

$$\begin{aligned} M_o &= W/4 \times a^2 / (a^2 - b^2) (1 + b^2/r^2) [ \text{Loge } a/c + 1/2 - c^2/2a^2 ] \\ &= 27068 \text{kg-m} \end{aligned}$$

$$\text{Net M} = 27068 - 24482 = 2586 \text{kg-m}$$

#### B. Section at the inner edge of the ring beam : $r=2.505\text{ m}$

Case I : forces of upward soil pressure on slab

$$\begin{aligned} M_o &= pr^2/16 + pb^2/4 [(\log_e r/a + 3/4(-1/3 + a^2/b^2 + a^2/r^2))] - [(a^2+r^2)/r^2, b^2/(a^2-b^2) \log_e a/b] \\ &= 18662 \text{ kg-m} \end{aligned}$$

$$M_r = -3pr^2/16 + pb^2/4 [(\log_e r/a + 3/4(1 + a^2/b^2 - a^2/r^2))] + [(a^2-r^2)/r^2, b^2/(a^2-b^2) \log_e a/b]$$

Using M-20 mix in foundation slab, beam etc.

$$\text{Effective depth required} = \sqrt{(2586/9.1)} = 16.85 \text{cm}$$

Say provide 23cm thickness on inner side of beam throughout and reduced to 20cm On the outer end of slab.

### RADIAL STEEL:

$$\text{Area of steel required} = 2199 \times 100 / 2300 \times 0.9 \times 17.4 = 6.11 \text{cm}^2$$

$$\text{Provide } 12 \text{mm dia. Radial bars @ } 1.13 \times 100 / 6.11 = 18.49 \text{cm c/c}$$

$$\text{Provide @ } 183.0 \text{mm C/C (86no.)}$$

$$\text{At top } 50\% \text{ Steel: } 8 \text{mm dia. @ } 0.5 \times 100 / 3.06 = 16.34 \text{ C/c}$$

$$\text{Provide @ } 162.3 \text{mm C/c ( 97 no.)}$$

### CIRCUMFERENTIAL STEEL:

Ast (on the inner edge of foundation slab)

$$= 2586 \times 100 / 2300 \times 0.9 \times 16.2 = 7.71 \text{cm}^2$$

$$\text{Spacing of } 12 \text{mm dia. Rings} = 1.13 \times 100 / 7.71 = 14.65 \text{ cm C/c}$$

Ast ( on the inner edge of Foundation Beam )

$$= 2133 \times 100 / 2300 \times 0.9 \times 16.2 = 6.37 \text{ cm}^2$$

Spacing of 12mm dia. Rings =  $1.13 \times 100 / 6.37 = 17.74 \text{ cm C/c}$

Provide 5, 12mm Dia. Rings of inner side of beam at bottom with spacing 17.5cm C/c near the inner edge slab to 14.5 cm C/c near inner edge of beam.

#### **ON OUTER SIDE OF BEAM :**

$$\text{Ast} = 2211 \times 100 / 2300 \times 0.9 \times 16.2 = 6.60 \text{ cm}^2$$

Spacing of 12mm dia. Rings required =  $1.31 \times 100 / 6.60 = 17.12 \text{ cm c/c}$

$$\text{One outer edge of slab Ast} = 1879 \times 100 / 2300 \times 0.9 \times 13.2 = 6.88 \text{ cm}^2$$

Spacing of 12mm dia. Rings required =  $1.31 \times 100 / 6.88 = 16.42 \text{ cm c/c}$

Provide 5 rings 12 mm dia. On outer side of beam at bottom.

#### **AT TOP : CIRCUMFERENTIAL STEEL :**

At top 50% steel on inner edge of beam steel required =  $3.19 \text{ cm}^2$

On inner edge of slab =  $3.86 \text{ cm}^2$

Provide 10mm dia. rings at 24cm c/c near beam to 20cm c/c near inner edge of slab.

Provide 4 rings 10mm dia. At top on inner side of beam

Ast ( on outer side of foundation beam )

Steel required at top near outer edge of beam =  $3.30 \text{ cm}^2$

Steel required at top near outer edge of slab =  $3.34 \text{ cm}^2$

So provide 10mm dia. steel rings @ 23cm c/c near beam to 22 cm c/c near outer edge of slab i.e provide 10 mm dia. on outer side of beam.

#### **References**

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