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ANALYSIS & DESIGN OF OVERHEAD SERVICE RESERVOIR (For 75,000 Liter Capacity)

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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/m3 of concrete. It should also be less than 530 kg/m3 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.

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 kg/m^3 of concrete. It should also be less than 530 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.

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.

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^2	**N/mm^2**	**N/mm^2**
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, reinforcemeat acting in conjunction with diagonal compression ic 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 kg/cm^2 = 1 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 eater-logged soils unto 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})	125 125	175 175

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 $300*10^{-6}$ 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 linking, 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 direstions 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 r 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 ore 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 cone fete 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 = $(2.505^2 + 1.15^2)/2x1.15=3.303m$

VOl. occupied by bottom dome=$3.142x1.15^2(3.303-1.15/3)=12.13m^3$

If height of water is ‘h’ m.then $3.141x5.59^2$ x h/4=75+12.13;h=3.55m

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 x 1.0=4.434m

Sin Ø = 2.805/4.434 = 0.633 ; Cos Ø = 3.434/4.434=0.775

Dead load =262.5 kg/m^2

Thrust at edges =262.5x4.434/ (1+.755)=656.0 kg/m

Compressive street = 656.0/7.5x100=.875 kg/cm^2

Minimum steel = .24% of concrete area = .24x7.5x100/100=1.80cm^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 450m.provide6mm 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 = 656x0.755 =509 kg/m

Hoop tension = 509x5.70/2 = 1451kg

Area of steel bars = 1451756/1500 = .97cm^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

= 15x10+12x4x.5 = 174cm^2

Tensile stress = 1451/174 = 8.34kg/cm^2 < 12kg/cm^2 (safe)

Dead load steps = 0.91x0.2635x0.1939x2500x13/2=756kg

W.m.rR.= 1000x5.59x3.55/2=9923

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

Depth	Coeff	Hoopforse	Steel required	Steel provided
0.0H	0.002	20	0.02 Cm^2	8mmØrings @ 252 mm C/con each
0.1H	0.100	993	0.67 Cm^2	face in 1260mm (10no rings = 5+5)
0.2H	0.198	1965	1.31 Cm^2	
0.3H	0.299	2967	1.98 0. Cm^2	4H 0.403 3999 2.67 Cm^2 8mmØ
				rings @ 280mm C/c on each
0.5H	0.521	5170	3.45 Cm^2	face in 840 mm(6no rings =3+3)
0.6H	0.650	6450	4.30 Cm^2	
0.7H	0.764	7581	5.06 Cm^2	8mmØ rings @ 190 mm C/c on each
0.8H	0.776	7701	5.14 Cm^2	face up to 1900 mm (20 rings = 10+10)
0.9H	0.536	5319	3.55 Cm^2	

…………………………………………………………………………………………………

Minimum steel required = 0.24% = 2.64 Cm^2

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

Bending moment = 0.0079x 1.000x3.555^3=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^2

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^2

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

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^2

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^2

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 = 0.137x7648x2.705^2x3.141/2=12045kg-m

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

Maximum twisting moment = 0.021x7648x2.705^2x3.141/2=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^2

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

Provide6,20mmØat top = 18.85cm^2 curtail 2,20mmØat 8.30mfrom column edge.

Provide4,16mmØ+1.10mmØat bottom throughout = 8.82cm^2

Side face reinforcement = 0.1x40x65= 2.60cm^{2}i.e.1.30cm^{2}2,10mmØon each side.

Ast at supports = 18.85+3.142+8.82=30.81 cm^2 >181.09+7.97=26.06 cm^2

Ast at-mid span = 12.56+3.142+8.82=24.53 cm$^{2>}$8.64+7.97=16.61 cm^2

S.F. at edge of coloum7648x3.142x5.41/8-7648x0.2=14721kg.

S.F. at point of max torsion = 2.705(45^{0}—19.25^{0})3.141x7648/180=9300

S.F. at due to torsion = 1.6x1847/0.4=7388kg.

Total S.F. Point of maximum torsion = 9300+7388=16688kg.>14721kg100

As/bd = 100x8.82/40x61.0=0.36

Allowable shear stress tc = 2.45kg./cm^2

V-Cc bd = 16688-2.45x40x61=10710

Spacing of 10mmØ 2 legged stirrups = 2x1.13x1500x61/1071=14.350 cm C/c

So provide 2 legged. 10mmØ stirrups @ 145mm c/c throughout

Gallery : Self Wt. of 10cm slab = 25-kg/cm^2

L.L .= 300.0 kg/m^2 Total Load = 550kg/ cm^2

B.M. = 550x0.9^2/2=223kg/m

De = 223x100/9.1x100=4.95cm

Provide, D = 100mm(de=65mm)

At = 223.0x100/1900x0.892x6.5=2.03 cm^2

Minimum steel required = 0.24%=2.4 cm^2

Spacing of 8mmØbars = 0.5x100/2.4=20/0cm c/c

Provide 8 mm Ø radial bars @ 200 c/cand 5 Nos. rings 8 mm Ø as shown in drawing.

Parapet Wall

Provide 100 mm thick parapet wall reinforce with 8 mm Ø @ 200 c/c both ways as shown.

Max vertical load in extreme columns = 112.83/5.41=20856kg

Direct stress = 69856/1387.31 = 50.36kg/cm^2

Seismic shear per column = 1/4x5140=1285kg

B.M. in column = 1767x100/7057.8=25.04kg/ cm^2

bending stress = 1767x100/7057.8=25.04kg/ cm^2

Check For

<u>stresses :</u> 50.36/60x1.333+25.04/85x1.333=0.63+0.23=0.86<1.00

Tank Empty Case

Equivalent weight W = 151.50-75=76.50T

Time Period = 2x3.142√ (76.50/9.81x662) = 0.68sec sag = 0.14

Horizontal seismic coeff. = 1.0x1.5x0.2x0.14=0.042

Horizontal seismic force = 0.042x76.50=3.22T

Turning moment of wind above foundation top = 3.22x21.95= 70.68 t-m

Max. vertical load in extreme column = 70680/5.41=13065kg

Check For Stresses

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

Direct shear stress in column= 17185/1387.31= 12.39 kg/cm

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

4/4= 27 kg/cm^2 (o.k safe)

Design of braces: Seismic shear in each bay = 1285kg

Wind shear in bottom bay= 1742kg

Wind shear in second bay= ¼ (3153+3/4x1423+2391x14.55/16.55)=1581kg

Wind shear in third bay= ¼ (3153 +2/4x1423+2391x14.55/16.55)=1378kg

For 4th & 5th bay seismic shear will be dominating

B, Min 1st brace from bottom = (1742x2.975/2+1581x3.60/2)= 5437 kg-m

B,M in 2nd brace from bottom= (1581x3.60/2+1378x4.25/2)=5775 kg-m

B,M in 3rd brace from bottom= (1378x4.25/2+1285x4.25/2)=5659 kg-m

B,M in 3rd and 4th brace from bottom= 1285x4.25=5462 kg-m

Design moment= 5775kg-m

Torsional moment=5%=289 kg-m

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

Total B.M=5775+553=6328 kg-m

Ast=6328x100/2300x37x1.333=5.58 cm^2

Provide 3,16 dia at top and bottom = 6.03cm^2

Total shear force= 2x5775/3.42+1.6x289/0.20=5690kg

100 Ast/bd=100x6.03/20x37=0.82Cc=3.6kg/cm2

Shear force taken by concrete=3.6x20x37=2664kg

Sacing of 8mmdia D legged stirrups=2x0.5x2300x37/(5690-2664)=28.12cm c/c

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

Design Of Foundation

Load transferred through 4 column= 49000x4=196000kg

Add 10% wt. of foundation= 20000kg

Total load= 216000kg

Area of foundation required= 216/8.75= 24.69 M^2

Wind pressure on tank portion and ring beam=

(4.15+9+.65)x5.81x.7x136=3153kg

Acting at 19.10+5.70/2=21.95m above foundation top

Wind force on coloumns=16.55x3x0.40x0.7x136=1891kg at

18.35/2+0.75=9.925m above beam

Wind load on braces=4x0.45x5.81x136=1423kg

Wind load on stairs=500kg

Total wind forc on columns and braces=1891+1423+500=3814kg.acting

At 9.925m above F beam

Overturning moment about top of F

Beam=3814x9.925+3153x21.95=107063kg-m

Max load in the remotest column=107063/5.41=19790kg

Tank Full:

Max. total load=49000+19790=68790kg

Direct stress= 68790/1387.31=49.59 kg/cm^2

B.M in column=1742x2.75/2=2396kg-m

Bending stress=2396x100/7057.8=33.95kg/cm^2

Check for stress=49.59/60x1.333+33.95/85x1.333=0.63+0.30=0.93<1(ok)

Tank Empty: minimum axial load=49000-19790-75000/4=10460kg

Direct stress=10460/1387.31=7.54kg/cm^2

Net tensile stress=33.95-7.54=26.41kg/cm^2<3/4x27x1.33=27kg/cm^2(ok)

Analysis Due To Seismic Forces:

Tank full: Wt. of main tank =130.0T

Wt. of staging W2= 4x6.00+12.32028=64.32T

Equivalent wt.= w^1+w2/3=30.0+64.32/3=151.50

K=4x12EI/13=4x12x2085x10^6 x 141156x10/(2.975^3 +3.60^3 +4.253+4.02533)=662t/m

Time period=2x3.142 (151.50/9.81x662)=662t/m

Time period=2x3.142(151.50/9.81X662)=.96sec

Sa/g=0.113

The tank is to be constructed in seismic zone 3

Horizontal seismic coeff.=1.0x1.5x0.2x0.113=.0339

Horizontal seismic force=0.0339x151.50=5.14t acting at 21.95m above the F.beam

Turning moment about foundation top = 5.14x21.95=112.83t-m

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

A=3.142/4(7.15^2-3.672)=29.57m^2

I=3.142/64(7.15^4-3.67^4)=119.40m^2

Z=119.40/3.575=33.39m^3

Check For Wind Forces:

O. moment due to wind forces = 107063+0.8x(3153+3814)=112637kg-m

Bending pressure due to wind pressure = 112.64/33.39=3.38 t/m^2

Direct pressure = 7.31+3.38=10.69 t/m^2 <1.25x8.750 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+5140x0.8=116942kg-m
Bending pressure due to wind pressure =116.95/33.39=3.51 t/m^2
Direct pressure = 216.0/29.57=7.31 t/m^2
Total pressure = 7.31+3.51= 10.82 t/m^2 <1.33x8.750t/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 = 196.0/3.142x5.41=11531kg.
Maximum B.M. at support = 0.137x11531x2.705^{2}x3.142/2=18.160t-m
Maximum B.M. at mid span = 0.07x18.16/0.137=9.28t-m
Maximum tensional moment = 0.021x18.16/0.137=2.784t-m
Equivalent B.M. due to torsion = 2784(1+800/400)/1.7=4913kg-m
Total – ve B,m. at support = 18160+4913=23073
Total – ve B,m. at mid span = 9280+4913=14193kg-m
Effective depth required = 18160x100x1000/9.1x40=70.6cm
So provided depth = 80.0cm is O.k.
Ast at support = 23073x100x1000/2300x0.9x74.0=15.07 cm^2
Ast at mid span = 14193x100x1000/2300x0.9x74.0=9.27 cm^2
Provide 5,20 mm Ø at bottom Ast = 15.70 cm^2 out of which curtail 2,20mm Ø at 1.3 m from support
At top, provide 3,20mmØ = 9.42cm2 provide 1,16 mmØ on each face of ring beam
S.F. at face of column = 11531x3.14x5.41/8-11531x0.2=22195kg
S.F. at point of contra flecure =3.142x5.41x25.75^{0}x11531/180x2=14020kg-m
Torsional shear = 1.6t/b = 11136kg Design shear = 25156kg
100 As/bd = 100x9.42/40x74=2.40kg/cm^2
Shear taken by concrete = 2.40x40x74=7104kg
Balance shear = V-Cc bd= 25156-7104=18052kg-m
Spacing of 10mm Ø 2 legged strriups = 2x0.785x300x74/18052=14.80CmC/c
Provide 2 legged 10mm Ø shear stirrups @ 145 mm C/c throughout.

Design of stair case:

Total height = 22.0-3.0=19.0m
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^2

(b) DESIGN FOR VERTICAL LOAD:

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

Case II. Forces of downward ring beam load on slab

M_0=w x a^2 $(1+b^2)$ [Loge a/c + ½-c^2]
4 a^2- b^2 r^2 $2a^2$

=6271kg-m

Net M_0=20795-18662=2133kg-m

M_0 = 6271-4072=2199kg-m

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

Case I. Forces of upward soil pressure on slab : M_r =0

M_0 = -pr^2+pb^2 (loge r+3/4(-1/3+a^2+ a^2)-($a^{2+}r^2$.b^2 $\log_e$ a/b)

=13901kg-m

Case II. Forces of downward ring beam load on slab:- r = a = 3.575_m

M_0 = w (loge c + ½ +a_2 (r_2+b_2) [Log_e a/c + ½-c_2]-c_2)
4 r a_2- b_2 r_2 $2a_2$ 2 r_2

=15780kg-m

Net M_0 = 15780-13901 = 1879kg-m

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

Case I. Forces of upward soil pressure on slab

M_0 = -pr^2 + pb^2 ($\log_e$ r+¾(-1/3+a^2+ a^2) -($a^{2+}r^2$, b^2 loge a/d)
16 4 a b2 r2 r2 a2-b2

Mr = - 3 pr^2 + pb^2 ($\log_e$ r+¾(1+a^2+ a^2) -($a^{2+}r^2$, b^2 loge a/d)
16 4 a b^2 r^2 r^2 a^2-b^2

=3590 kg – m

<u>**STAGING :**</u> 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 = 3.142/4x0.4x0.4x19.10x2500=6.00t
Wt. of braces = 4x3.42x0.20x0.45x2500=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 = 49x1000/60=817cm^2
Area provided = 3.142x40x40/4=1256.8cm^2 is O.K.
Area of steel required = 0.008x817=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(40x40)+(16.5x7.91)=1387.31cm^2
I= 141156cm^4
Section modulus = 141156/20= 7057.85 cm^3

ANALAYSIS 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^2)	Pz(kg/cm^2)
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 = 127.3x10+132.60x5+142.05x5+149.70x4.05/24.05=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 = 4.69x0.91x0.16x2500=1708kg.
live load 500kg,total load = 2964kg
B.M=2964x3.83/10=1136kg-m
De=$\sqrt{[1136\text{x}100/(91\text{x}91)]}$=11.71cm
Provide D=16.0cm
Area of steel = 1136x100/2300x0.90x12.5=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= 2/3x0.9=0.6m
B.M=3664x0.60=2199kg-m
Effective depth required = $\sqrt{(2199\text{x}100/9.1\text{x}40)}$=24.58cm

So provide over all depth D=32cm reduced to16 cm near outer edge

Ast required =2199x100/2300x0.90x28.5=3.73cm^2

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

DESIGN OF FOUNDATION SLAB BY PLATE LOAD THEORY:

Intensity of pressure (p)=10.69/1.25=8552kg/Cm^2 say 8600 kg/ Cm^2

Outer radius of foundation slab (a) =3.575m

Inner radius of foundation slab (b) =1.835m

Central radius of foundation slab (c) =2.705m

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

r = 1.835m

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

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

= 24482 kg-m

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

W = 8600x29.57=254302 kg

Mo = $W/4x\ a^2/(a^2-b^2)\ (1+b^2/r^2)$ [Loge a/c + ½-$c^2/2a^2$]

=27068kg-m

Net M = 27068-24482=2586kg-m

B. <u>Section at the inner edge of the ring beam :</u> r =2.505 m

Case I : forces of upward soil pressure on slab

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

=18662 kg-m

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

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

Effective depth required = √(2586/9.1)=16.85cm

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

RADIAL STEEL:

Area of steel required = 2199x100/2300x0.9x17.4=6.11cm^2

Provide 12mm dia. Radial bars @ 1.13x100/6.11=18.49cm c/c

Provide @ 183.0mm C/C (86no.)

At top 50% Steel: 8mm dia. @ 0.5x100/3.06=16.34 C/c

Provide @ 162.3mm C/c (97 no.)

CIRCUMFERENTIAL STEEL:

Ast (on the inner edge of foundation slab)

= 2586x100/2300x0.9x16.2=7.71cm^2

Spacing of 12mm dia. Rings = 1.13x100/7.71=14.65 cm C/c

Ast (on the inner edge of Foundation Beam)

= 2133x100/2300x0.9x16.2=6.37cm^2

Spacing of 12mm dia. Rings = 1.13x100/6.37=17.74cm 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 :

Ast = 2211x100/2300x0.9x16.2=6.60cm^2

Spacing of 12mm dia. Rings required = 1.31x100/6.60=17.12 cm c/c

One outer edge of slab Ast = 1879x100/2300x0.9x13.2=6.88cm^2

Spacing of 12mm dia. Rings required = 1.31x100/6.88=16.42 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 = 3.19cm^2

On inner edge of slab = 3.86cm^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= 3.30cm^2

Steel required at top near outer edge of slab = 3.34cm^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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