REV. NO.	DESCRIPTION	DESIGNED		CHECKED		APPROVED	
B	Revised as per client comments	AKHB	13.02.17	RR	13.02.17	BRJ	13.02.17
A	For Approval	KRC	04.05.16	RR	07.05.16	BRJ	09.05.16

REVISIONS

LARSEN & TOUBRO LIMITED
CONSTRUCTION DIVISION
Water,Smart World & Communication IC

CLIENT:	CONSULTANT :
TELANGANA DRINKING WATER SUPPLY PROJECT, GOVERNMENT OF TELANGANA	

PROJECT :	Providing drinking water to habitations in Komarambheem-Asifabad Segment in Adilabad District
SUPPLIER / CONTRACTOR	L&T CONSTRUCTION Water & Effluent Treatment SBG

JOB Ref. No. : LE150883

	NAME	SIGN	DATE
DSGN	KRC	KRC	04.05.16
CHKD	RR	RR	07.05.16
APPD	BRJ	BRJ	09.05.16

TITLE :

20 KL capacity OHBR - 30m staging height - Design Calculations

DOC./DRG. No.	SIZE	REV.
LE150883-C-WS-CW-DC-3015	A4	B

RELEASED FOR: ☐ PRELIMINARY ☐ INFORMATION ☑ APPROVAL ☐ CONSTRUCTION

LARSEN & TOUBRO LIMITED
ECC Division – EDRC

PROJECT:	Providing drinking water to habitations in Komarambheem-Asifabad Segment in Adilabad District	DOCUMENT NO. LE150883-C-WS-CW-DC-3015.		DATE 13-02-2016
TITLE	20 KL Capacity OHBR - Design Calculations	DESIGNED AKHB	CHECKED RR	PAGE

CONTENTS

Design of Over head Reservior

(1)	**DATA:**		
	Capacity of Tank	20	m^3
	Unit weight of RCC=	25	kN/m^3
	Unit weight of PCC=	24	kN/m^3
	Unit weight of soil =	18	kN/m^3
	Unit weight of sand filling inside bottom of shaft =	18	kN/m^3
	Unit weight of water=	10	kN/m^3
	Staging Height	30	m
	Net S.B.C of Soil =	150	kN/m^2
(2)	**PERMISSIBLE STRESS:**		
	Grade of concrete; f_{ck} =	M30	N/mm^2
	Grade of steel; f_y =	Fe500	N/mm^2
Ref Table 1 of IS:3370	Allowable stress as per IS:3370 relating to resistance to cracking		
	Allowable direct tensile stress in concrete σ_{at} =	1.5	N/mm^2
	Allowable bending tensile stress in concrete σ_{bt} =	2.0	N/mm^2
Ref Table 4 of IS:3370	Allowable stress in steel under direct tension, bending & shear = σ_{st} =	130	N/mm^2
	Allowable stress in steel under direct compression = σ_{sc}=	140	N/mm^2
	σ_{st2} =	150	N/mm^2
IS 456:200	Allowable stress in steel under direct tension, bending & shear = σ_{st} =	230	N/mm^2
	Allowable stresses as per IS:456 for strength calculations		
Ref Table 21 of IS:456	Allowable direct compressive stress in concrete σ_{cc} =	8	N/mm^2
	Allowable bending compressive stress in concrete σ_{cbc} =	10	N/mm^2
	Modular ratio = $m = \frac{280}{3\sigma_{cbc}} =$ m =	9.33	
	Neutral axis co-efficient; $n = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} =$ n =	0.42	
	Lever arm coefficient; j = 1-n/3 = j =	0.86	
	Moment coefficient = $K=0.5 \times \sigma_{cbc} \times (n \times j)$ =	1.81	N/mm^2
(3)	**Volume calculation**		
	Diameter of tank, D =	5.00	m
	Rise of Top Dome, h = =D/5 =5/5	1.00	m

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	Diameter of supporting shaft = D =	5.00	m
	Rise of bottom dome , h = =D/5 =5/5	1.00	m
	Height of water column in cylindtical portion of tank, H =	2.00	m
	Free board, F.B =	0.30	m
	Total Height of tank wall = H+FB-(1.8-h) =2+0.3+(1.8-1)	3.10	m
	C/C Diameter of internal shaft	1.20	m
	Outer Diameter of Internal shaft = (Dia+thk of wall)= 1.2+0.2	1.40	m
	Radius of Inner Shaft = =1.2/2	0.60	m
	Total height of Internal shaft = H-h+FB= =2-1+0.3	1.30	m
	Inner diameter of the tank = D-shaft thk+(wall thk/2) =5-0.25+(0.25/2)	4.75	m
	Volume of Cylindrical portion $=V_1= (\pi/4)\times(\text{ inner dia})^2\times H =$ $(\pi/4)\times(4.75)$^2×2	35.44	m^3
	Radius of curvature of bottom dome = R $=[(D/2)^2+h^2]/(2h)$		
	=[(5/2)^2+1^2)]/(2×1)	3.63	m
	Volume of bottom dome $=V_2= (\pi/3)\times(r^2\times(3R-h))$		
	=(π/3)×(1^2×(3×3.63-1)	10.36	m^3
	Volume of internal shaft $=V_3= (\pi/4)\times(dia^2\times (H-h))$		
	=(π/4)×[1.4^2×(2-1)]	1.54	m^3
	Total volume of tank without free board = $V_1-V_2-V_3$ =35.44-10.36-1.54	23.54	m^3
		OK	
	Total volume of tank with free board =	28.40	m^3
(4)	**Design of Top dome:**		

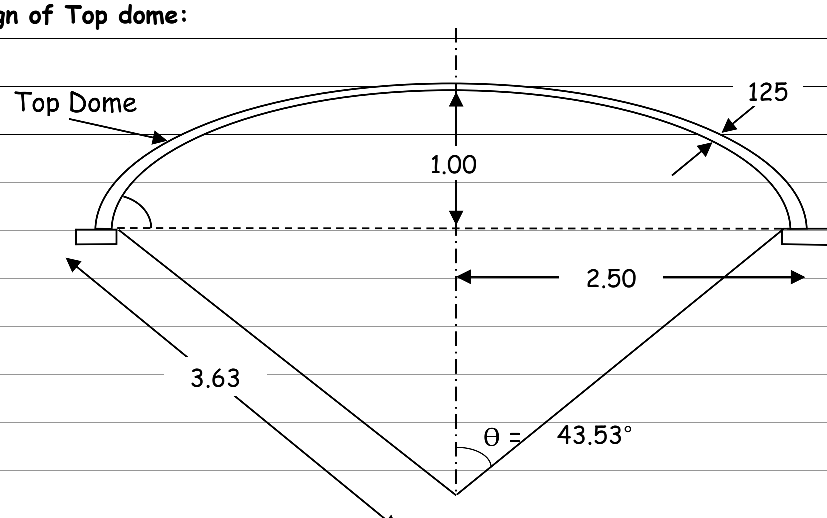

Figure 2: Top Dome.

	Radius of the chord, r = 5/2	2.50	m

	Rise of the top dome, h =	1.00	m
	Radius of the shell surface = $(r^2 + h^2)/2h$ = (2.5^2+1^2)/(2*1)	3.63	m
	Semi-central angle is given by		
	Sin θ = r_3/R = 0.69 that is, θ =	43.53°	
	=	0.760	rad
	Thicknes of the dome =	125	mm
	Self weight of dome (wg) = 0.125 X 25	3.125	kN/m^2
	Live load w_l =	1.50	kN/m^2
	Total load, w = = 1.5 + 3.125 =	4.63	kN/m^2
	Weight of the dome = $2\pi Rhw_g$ = 2π x 3.63 x 1 x 3.125 =	71.27	kN
	Live load on the dome = $2\pi Rhw_l$ = 2π x 3.63 x 1 x 1.5 =	34.21	kN
	Total load on top dome = 71.27 + 34.21 =	105.48	kN
	Meridional thrust = N_o = (wR)/(1+cos θ) =	9.73	kN/m
	Meridional Stress = 0.00973/0.125 =	0.08	MPa
	0.08 < 1.5 (OK)		
	As the stress is only nominal, provide the min. reinforcement of	0.24	%
	A_{sm} = 0.24×(125)×(1000)/100	300.00	mm^2/m
	Dia of bar =	10	
	Spacing of bar required =	260	mm
	Provide 10 mm dia bar @ 125 mm c/c in meridional direction		
	Circumferential force = wR[cos θ-(1/(1+cos θ)] =	2.44	kN/m
	Hoop stress = 0.00244/0.0015	0.02	MPa
	0.02 < 1.5 (OK)		
	As the stress is only nominal, provide the min. reinforcement of	0.24	%
	A_{sm} = 0.24×(125)×(1000)/100	300.00	mm^2/m
	Dia of bar =	10	mm
	Spacing of bar required =	260	mm
	Provide 10 mm dia bar @ 125 mm c/c in circumferential direction		
(5)	**Design of beam at balcony level and balcony slab**		
	<u>Design of balcony</u>		
	Clear width of walkway	0.75	m
	Width of beam at this level	350	mm
	Cantilever span of balcony from beam	0.40	m

	Thickness of slab	150	mm
	Self weight of slab =(0.15)×25×0.4=	1.50	kN/m
	Live load on slab	1.50	kN/m^2
	Load due to finishes	1.20	kN/m^2
	Total load acting on the walkway slab =0.15x25+1.5+1.2=	6.45	kN/m^2
	Max BM at Support =6.45x0.4²/2=	0.52	kN-m
	Effective Depth required = $\sqrt{(BM\times10^6)/(k\times1000)}$ = √((0.52×10^6)/(1.81×1000))	16.97	mm
	Provided 150 mm uniform thickness for walkway slab		
	Cover to the reinforcement	25	mm
	Diameter of bar	12	mm
	effective depth provided =150-25-12	119	mm
	Area of steel required =(0.52×10^6)/(0.86×119×130)	39.09	mm^2/m
	Minimum percentage of steel required =	0.24	%
	Minimum Area of steel required on center of slab = =0.0024×150×1000 =	360.00	mm^2/m
	Spacing of 12 mm dia steel =	250	mm c/c
	Spacing provided	200	mm c/c
	Area of steel provided =	565.49	mm^2/m
	percentage of steel provided =	0.48	
	Diameter of distribution bar =	10	mm
	Spacing of 10 mm dia tor steel =	200	mm c/c
	10 mm dia tor steel @ 200 mm c/c as distribution steel		
	Provide 12 mm main bar @ 200 mm c/c		
	Total weight of slab = 2×π×(5/2+350/1000+0.4/2)×0.4×(150/1000)×25	28.75	kN
(6)	**Design of Top ring Beam**		
	Hoop thrust on ring beam is same as the horizontal component of		
	the meridional thrust from the top dome. The hoop tension in the		
	ring beam is, therefore, equal to		
	Hoop Tension = $T = N_o \cos(\theta) R$ =	17.64	kN
	Where R =	2.50	m
	Size of the web of the ring beam:		
	b=	350	mm
	D=	300	mm

	Area of tension steel required, As=	=(17.64×1000)/130	135.692	mm^2
	Minimum percentage of steel =		0.24	%
	Minimum steel A_{min} =	=(0.0024)×350×300	252.00	mm^2
	Cover to the reinforcement =		25	mm
	Dia of bar =		16	mm
	Number of bars required		2	Nos.
	Number of bars provided		3	Nos.
	Area of steel provided =		603	mm^2
	Stress in concrete = $T/[A_g + (m-1)A_{st}]$ =			
	= (17.64x1000)/[(350x300)+(9.33-1)x603.19] =		0.16	N/mm^2
		0.16 < 1.5 (Safe)		
	Provide a ring beam of size 350 mm by 300 mm.			
	Provide 3Y16 at top and 3Y16 at bottom			
	Provide 8 mm dia stirrups at 250 mm centres.			
	Self weight of beam = 2π (2.675) (0.35 X 0.3) (25) =		44.12	kN
(7)	**Design of vertical wall of tank**			
	Total Wall height =		3.10	m
	height of water column =	=2+0.3 =	2.30	m
	Radius of tank		2.50	m
	Hoop tension, T = unit weight of water x H x D/2	=10x2.3x2.5	57.5	kN/m
	Thicknesss of wall =		250	mm
	H^2/Dt=	=3.1^2/(4.75x0.25)	8.09	
	Calculating tension and moment from IS 3770 Part 4			
From IS 3370	Hoop tension for hinged base and top free			
	Coefficient from table 9 of IS 3370 Part 4		0.76456	
	Hoop tension =coefficient x w x H x R	=0.76x10x3.1x2.5	59.2531	kN/m
	Maximum Hoop tension, T =		59.3	kN/m
	Ast required on each face for max tension =	=59.2530736842105×1000/(	227.90	mm^2
	Minimum Ast required as per IS 3370		0.24	%
	Ast minimum required on each face	=(0.0024×1000×250)/2	300	mm^2
	Dia of bar provided =		10	mm
	Spacing required on each face		250	mm

	Provide 10 mm dia @ 145 mm centres on both faces		
	Area of steel provided	541.65	mm^2
	Stress in concrete = $T/[A_g + (m-1)A_{st}]$ =		
	=(57.5×1000)/(1000×250+(9.33-1)×541.65)	0.23	N/mm^2
	0.23 < 1.5 (Safe)		
	Vertical Steel		
From IS 3370	Vertical Moment for Fixed base and top free		
	Coefficient from table 10 of IS 3370 Part 4	0.00842	
	Moment =coefficient x w x H^3 =0.0084x10x3.1^3	2.50975	kN-m
	Area of steel required for moment		
	=2.51x10^6/(130x(250-45-12/2)x0.86)	112.807	mm^2
	Minimum area of steel on each face	300	mm^2
	Diameter of bar provided	12	mm
	Spacing required	250	mm
	Spacing provided	200	mm
	Provide 12 mm dia @ 200 mm centres on both faces		
	Area of steel provided =(π/4)×12^2(1000/250)	565.49	mm^2
	Total weight of cylindrical wall =2×π×2.5×3.1×0.25×25	304.34	kN
(8)	**Design of bottom dome and internal shaft**		

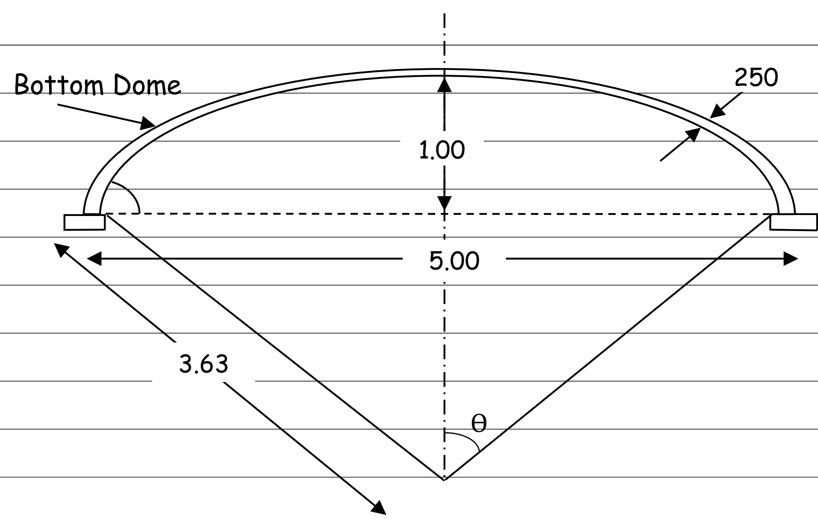

Figure 4: Bottom Dome.

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	Diameter at base of dome =	5.00	m
	Rise of bottom dome = h =	1.00	m
	Thickness of bottom dome, t =	250	mm
	Radius of the shell surface = (radius^2 + rise^2)/(2 x rise) =	3.63	m
	Weight of the dome slab = 2 x π x 3.63 x 1 x 0.25 x 25 =	142.55	kN
	Thickness of walls of Internal shaft =	200	mm
	Total Projection of platform required at top of internal shaft	750	mm
	Thickness of platform	150	mm
	Internal diameter of vertical shaft = =(2×0.6)-0.2	1000	mm
	External diameter = 1000 + 2 x 200 =	1400	mm
	Weight of water over bottom dome = (with FB) =28.4×10	284.0	kN
	Weight of vertical shaft = =π×((1400-200)/1000)×(200/1000)×1.3×25	24.50	kN
	Weight of circular platform		
	=π×(1000/1000+750/1000)×(150/1000)×(750-200)/1000×25	11.34	kN
	Total weight on dome = =142.55+283.99+24.5+11.34	462.39	kN
	Load/unit area = w = =462.39/((π/4)×5^2)	23.55	kN/m^2
	Meridional thrust = T_1 = = wR/(1+cos Θ)	49.57	kN
	where, cos Θ =	0.725	rad
	Meridional stress = (49.57*1000)/(130*1000) =	0.381	N/mm^2
	0.381 < 8 (Safe)		
	Circumferential force = wR [cos Θ -(1/(1+cos Θ)] =	12.37	kN
	Hoop stress = (12.37*1000)/(130*1000) =	0.1	N/mm^2
	0.1 < 1.5 (Safe)		
	Provide minimum reinforcement of	0.24	%
	Minimum steel required, A_{st} =	600	mm^2
	Diameter of bar provided =	10	mm
	Spacing of bar required =	125	mm
	Provide 10 mm dia bar at 125 mm centres both radially and in circumferential direction.		
	Maximum hoop compression in the internal shaft =		
	=10 x 1.3 x ((1400 - 200)/1000)/2 =	7.8	kN
	Hoop stress = = (7.8 x 1000)/(130 x 1000) =	0.06	N/mm^2
	0.06 < 8 (Safe)		
	Provide minimum reinforcement of	0.24	%

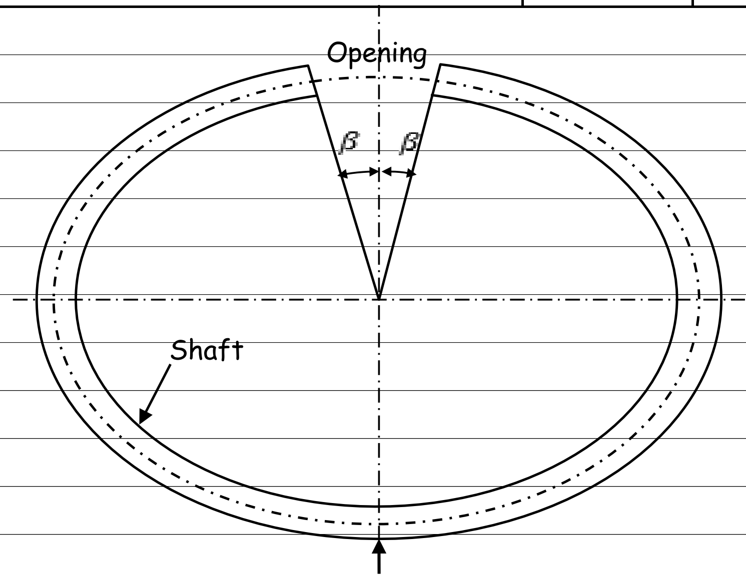
	Minimum steel required, A_{st} =	480	mm^2
	Diameter of bar provided =	10	mm
	Spacing of bar required =	160	mm
	Provide 10 mm dia bar at 160 mm centres in both directions.		
(9)	**Design of bottom ring beam**		
	Horizontal thrust from bottom dome = =49.57*COS(43.53)	35.91	kN
	Net Hoop Tension force in ring beam, H =	35.91	kN
	Hoop compression = = 35.91 (5/2) =	89.78	kN
	Dimensions of bottom ring beam :		
	b =	350	mm
	D=	400	mm
	Area of tension steel required =(89.775 x 1000)/130	690.577	mm^2
	Provide minimum reinforcement of	0.24	%
	Minimum steel required, A_{st} = =(0.24/100)×350×400	336	mm^2
	Diameter of bar provided =	16	mm
	Number of bars required =	6	Nos.
	Area of tension steel provided	1206	mm^2
	Stress in concrete = T/[Ag + (m-1)Ast] =		
	=(89.775×1000)/(350×400+(9.33-1)×1206.37	0.60	N/mm^2
	0.6 < 1.5 (Safe)		
	Provide a ring beam of size 350 mm by 400 mm.		
	Provide 3Y16 at top and 3Y16 at bottom		
	Provide 8 mm dia stirrups at 200 mm centres.		
	Weight of bottom ring beam = π x 5 x (0.35 x 0.4) x 25=	54.98	kN
(10)	**Design of supporting cylindrical shaft**		
	Centre to centre Diameter of shaft =	5.00	m
	Height of shaft (above G.L.) =	30	m
	Thickness of shaft wall above G.L. =	250	mm
	Minimum thickness of shaft required as per IS: 11682-1985	150	mm
	Total depth of foundation below G.L. =	3.00	m
	Deptht of shaft (below G.L.) = = 3 - 0.5 =	2.50	m
	Thickness of shaft wall below G.L. =	350	mm

	Self weight of shaft above G.L. = $\pi \times 5 \times 25 \times 30 \times 0.25$ =	2945.24	
	Self weight of shaft below G.L. = $\pi \times 5 \times 25 \times 2.5 \times 0.35$ =	343.61	kN
	Thickness of shaft wall above G.L. =	250	mm
	Loads acting on shaft at ground level:		
	(1) Top dome	105.48	kN
	(2) Top ring beam	44.12	kN
	(3) Balcony	28.75	kN
	(4) Tank wall	304.34	kN
	(5) Bottom spherical dome	142.55	kN
	(6) Internal shaft + platform	35.84	kN
	(7) Bottom ring beam	54.98	kN
	Weight of tank portion =	**716.06**	kN
	(8) Supporting shaft	3288.85	kN
	Total Dead load on top of footing =	**4004.91**	kN
	(9) Weight of water (Hydro test condition)=	283.99	kN
	(10) Weight of water (Working condition)=	235.45	kN
	Wind pressure:		
	Basic wind speed, V_b=	50	m/s
	Risk Coefficient, k_1=	1.08	
	Terrain, height and structure size factor, k_2=	1.11	
	Topography factor, k_3=	1	
	Design wind speed, $V_z = V_b \times k_1 \times k_2 \times k_3$ =	59.94	m/s
	$P_z = 0.6\ V_z^2$ =	2.16	kN/m^2
Ref Pg.	**Total moment due to wind load about base of footing , M**	**2840.45**	**kN-m**
Wind load	Area of cross section of shaft, A = $\pi\ [(2.625)^2 - (2.375)^2]$ =	3.93	m^2
calculation	Second moment of area, I :		
	$I = (\pi/4)\ [(2.625\text{^}4) - (2.375\text{^}4)]$ =	12.30	m^4
	Stress at base section:		
	Tank empty condition:		
	W =	4004.91	kN
	Outer dia of shaft, D =	5.35	m
	Mean radius of shaft, r =	2.5	m
	M =	2840.45	kN-m
	e = (M/W) =	0.71	m

	e/r = 0.71/2.5 =	0.284	m
	e/r <= 1/2 (OK)		
IS 11682-	***This section is under compression only***		
1985	σcv = (W/2π r t)[1 + (2e/r)] =	1.14	N/mm^2
	1.14 < 0.38x 30 (Safe)		
	Tank working condition + wind:		
	P =	4240.36	kN
	M =	2840.45	kN-m
	e= M/W =	0.67	m
	e/r = 0.67/2.5 =	0.27	
IS 11682-	**e/r <= 1/2 (OK)**		
1985	σcv = (W/2π r t)[1 + (2e/r)] =	1.18	N/mm^2
	1.18 < 0.38x 30 (Safe)		
	Tank Hydro test condition		
	W =	4288.90	kN
	M =	0	N-mm
	e = M/W =	0	
IS 11682-	σcv = (W/2π r t)[1 + (2e/r)] =	0.78	N/mm^2
1985	**0.78 < 0.38 x 30 (Safe)**		
IS 11682-	Provide minimum longitudnal reinforcement of	0.25	%
1985	Area of steel required on each face, A_{st} =	312.5	mm^2
	Diameter of bar provided =	12	mm
	>=10 mm (OK)		
	Spacing of bar required =	360	mm
	Spacing of bar provided =	200	mm
	Provide 12 mm dia bar at 200 mm centres vertically on each faces.		
	Area os steel provided on each face =	565.5	mm^2
	Circumferential reinforcement in shaft:		
IS 11682-	Provide minimum circumferential reinforcement of	0.2	%
1985	Area of steel required on each face, A_{st} =	250	mm^2
	Minimum steel required per meter length on each face =	200	mm^2
	Diameter of bar provided =	10	mm
	Spacing of bar required =	310	mm

	Spacing of bar provided =	200	mm
	Area of steel provided per metre length of shaft=	392.70	mm^2
		> 200 (OK)	
	Provide 10 mm dia bar at 200 mm centres circumferentially on each faces.		
	Area of steel provided =	392.7	mm^2
	Check for seismic forces		
	Height of staging above groung level =	30.00	m
	Stiffness of shaft, $k = 3\ EI/l^3$ =		
IS 456-	$E = 5000(f_{ck})^{0.5}$ =	27386.13	N/mm^2
2000	I = (π/4) [(2.625^4) - (2.375^4)] =	12.30	m^4
	l= length of staging =	30.00	m
	k=	37436.84	kN/m
	Seismic coefficient is given by : $A_h = \frac{Z\ I}{2\ R}\left(\frac{S_a}{g}\right)$		
IS: 1893-	where, Zone Factor, Z =	0.1	
2002	Importance Factor, I =	1.75	
	Response reduction Factor R =	3	
	Spectral Acceleration , (S_a/g)		
	Tank Empty condition :		
	Weight of tank Container =	716.06	kN
	Weight of 1/3 of staging = (1/3) x (2945.24) =	981.75	kN
	Seismic weight for tank empty condition, W_s=	1697.81	kN
	Time period when tank empty, T_e = $2\pi\ [(W_s/9.81)\ /\ k]^{0.5}$		
	= 2π x {(1697.81/9.81)/(37436.84)}^0.5=	0.43	sec
IS: 1893-	For rocky, or hard soil sites, corresponding S_a/g =	2.34	
2002	The design horizontal seismic coefficient, A_h =	0.07	
	Maximum horizontal seismic force acting at top of staging =	115.91	kN
	Moment due to seismic forces at top of footing:		
	Total load, W =	4004.91	kN
	Moment, M=	3767.20	kN-m
	e= M/W =	0.94	m
	e/r = 0.94/2.5 =	0.38	

IS 11682-	**e/r <= 1/2 (OK)**		
1985	σcv = (W/2π r t)[1 + (2e/r)] =	1.28	N/mm^2
	1.28 < 0.40 x 30 (Safe)		
	Tank Full condition :		
	Weight of tank Container =	716.06	kN
	Weight of 1/3 of staging = (1/3) x (2945.24) =	981.75	kN
	Weight of water =	235.45	kN
	Seismic weight for tank full condition =	1933.25	kN
	Time period when tank full, T =	0.46	sec
IS: 1893-	For rocky, or hard soil sites, corresponding Sa/g =	2.34078	
2002	The design horizontal seismic coefficient, A_h =	0.07	
	Maximum horizontal seismic force acting at top of staging =	131.99	kN
	Moment due to seismic forces at top of footing:		
	Total load, W =	4240.36	kN
	Moment, M= =131.99*(30+2.5)	4289.62	kN-m
	e= M/W =	1.01	
	e/r = 1.01/2.5 =	0.4	
IS 11682-	**e/r <= 1/2 (OK)**		
1985	σcv = (W/2π r t)[1 + (2e/r)] =	1.39	N/mm^2
	1.39 < 0.40 x 30 (Safe)		
	Check for stress at openings:		
	Size of opening : width =	1	m
	height =	2	m
	Maximum vertical compressive stress in concrete at outside diameter of shaft shell is given by :		
IS 11682-1985	$$\sigma_{cv} = \frac{W}{2(\pi-\beta)\,r\,t}\left[1 + \frac{2\left\{\frac{e}{r} + \frac{\sin\beta}{\pi-\beta}\right\}\{(\pi-\beta)\cos\beta + \sin\beta\}}{(\pi-\beta) - \frac{1}{2}\sin 2\beta - \frac{2\,sin^2\,\beta}{(\pi-\beta)}}\right]$$		



Where,

	ß = half the angle substended by neutral axis		
	as a chord on the circle of radius r =	0.20	rad
	W = Total vertical load above section under		
	consideration in N =	4240	KN
	M = Moment in vertical plane at the section		
	under consideration in N-mm =	4.29E+03	KN-m
	e = M/W =	1.012	m
	r = Mean radius of circular shaft in m =	2.5	m
	t = Thickness of shaft in mm =	250	mm
	e/r =	0.405	
IS 11682-1985	From Table 1	2.6515	N/mm^2
	2.7 < 0.40 x 30 (Safe)		
(11)	**Design of raft foundations**		
	Total load from tank and shaft = (Dead load on top of footing + weight of water working condition)		
	=4004.91KN+235.45KN - (a)	4240.36	kN
From staad	Total weight of staircase =	1296	kN
	Load from staircase = - (b)	1296	kN
	Diameter of raft slab, D_r =	8.4	m
	Thickness of raft slab, t =	500	mm
	Self weight of footing = $(\pi/4)\times D_r^2 \times t$ =		

	=(π/4)×8.4^2×0.5×25	- (c)	692.72	kN
	Weight of Earth filling inside the shaft upto G.L.			
	= [π (4.65²)/4] X 2.5 X 18 =	- (d)	764.20	kN
	Weight of earth filling over the raft slab upto G.L.			
	= [π (8.4²-5.35²)/4] X 2.5 X 18 =	- (e)	1482.19	kN
	Total load acting on raft slab, W =	=(a)+(B)+(c)+(D)+(e)	8475.47	kN
	Net S.B.C. of soil =		150	kN/m^2
	Gross S.B.C at depth of 3 m below G.L. (For normal load)=			
		=150+3×18	204	kN/m^2
	Gross S.B.C at depth of 2.5 m below G.L. (For seismic/wind load)=			
		=150×1.25×3×18	241.5	kN/m^2
	Area of footing, A =	=(π/4)×8.4^2	55.42	m^2
	Direct load, W =		8475.47	kN
	Moment M = (Tank full condition under seismic)		4289.62	kN-m
From staad	Moment from staircase column (seismic case) =		45.00	kN-m
	Total moment =		4334.62	kN-m
	Section modulas, Z=		58.19	m^4
	Maximum intensity of soil pressure at base = [W/A + M/Z] =		227.43	kN/m^2
		227.43 < 241.5 (Safe)		
	Minimum intensity of soil pressure at base = [W/A - M/Z] =		78.45	kN/m^2
		78.45 >0 (No tension)		
	Adopt Diameter of raft slab = 8.4 m			
	Projection of raft beyond face of shaft =		1.525	m
	Maximum net soil pressure, w =			
	=227.43-(500/1000*25)-(18*2.5)		169.93	kN/m^2
	The loading at base is taken as annular loading on the mean diameter of the shaft.			
	Diameter of raft slab = 2a =		8.4	m
	Diameter of the shaft = 2b =		5.00	m
	Radial moment at centre of foundation is given by:			
	$M_r = \frac{W}{8\pi}\left[2\,log_e\left(\frac{a}{b}\right)+1-\left(\frac{b}{a}\right)^2\right]-\frac{3}{16}w.a^2$	=	5.61	kN-m/m
	Moment at junction of footing and tank walls at a radius of 2.5 m is given by:			

$$M_{max} = \frac{W}{8\pi}\left[2\,log_e\left(\frac{a}{b}\right) + 1 - \left(\frac{b}{a}\right)^2\right] - \frac{3}{16}w\,(a^2 - b^2) = 204.74 \text{ kN-m/m}$$

Design ultimate moment = M_{ur} =	(1.5 x 204.74) =	307.11	kN-m/m
Effective depth required d = $[M_u/.133\ f_{ck}\ b]^{0.5}$ =		277.43	mm
Effective depth provided at the section =		442.00	mm
		(OK SAFE)	
Compute parameter:			
M_u/bd^2 =		1.572	
Refer Table-4 of SP : 16 and read out the percentage reinforcement as:			
p_t= 100 A_{st} / bd =		0.38672	
Area of steel required, Ast =		1709.30	mm^2/m
Diameter of bar provided =		16	mm
Cover to the reinforcement =		50	mm
Actual effective depth at the section =		442	
Spacing of bar required =		100	mm

Provide 16 mm dia bar at 100 mm centres both ways at bottom of footing.

Area of steel provided =		2010.62	mm^2/m
Design ultimate moment = M_{uc} =	(1.5 x 5.61) =	8.415	kN-m/m
Compute parameter:			
M_u/bd^2 =		0.04	
Refer Table-4 of SP : 16 and read out the percentage reinforcement as:			
p_t= 100 A_{st} / bd =		0.12	
Area of steel required, Ast =		532.80	mm^2/m
Diameter of bar provided =		12	mm
Cover to the reinforcement =		50	mm
Effective depth at the section =		444	
Spacing of bar required =		200	mm

Provide 12 mm dia bar at 200 mm centres both ways at top of footing.

Check for shear :

	Intensity of soil pressure, p =		169.93	kN/m^2
	Cantilever projection of raft =	(8.4-5-0.35)/2 =	1.525	m
	Max. SF at a distance 'd' from the support =	169.93*(1.525-0.442) =	184.03	kN
	Shear stress , t_v = V/bd =	184.03x 1000/1000(442) =	0.42	N/mm^2
	percentage steel at the section, 100As/bd =		0.387	%
	The allowable shear stress for 0.387 % tension reinforcement is			
		k x t_c =	0.44	N/mm^2
		Slab is safe in shear (OK)		

APPENDIX

(1)	**Stability Check -** Tank empty conditon			
	Wind force		182.30	kN
	Moment due to wind force		2840.45	kN-m
	Seismic force		115.91	kN
	Moment due to seismic force		3767.20	kN-m
	Max. horizontal force		182.30	kN
	Max. overturning moment = OM		3767.20	kN-m
	Total vertical DL			
	=(Top container(without water) + shaft + stair case + raft + earth inside and outside)		8240.03	kN
	0.9 DL	=0.9 x 8240.03	7416.02	kN
	Restoring moment = RM =DL x (raft dia)/2	=8240.03 x 8.4/2	34608.11	kN-m
	Check for safety against overturning			
	Factor of Safety =OM/RM	= 3767.2/34608.11 =	9.19	
			>1.5 safe Ok	
	Check for safety against sliding			
	Factor of Safety =(0.9DL x μ)/(Max horizontal force)	=7416x0.4/182	16.27	
			>1.25 safe Ok	
(2)	**Stability Check -** Tank full conditon			
	Seismic force		131.99	kN
	Moment due to seismic force		4289.62	kN-m
	Max. horizontal force		182.30	kN
	Max. overturning moment = OM		4289.62	kN-m
	Total vertical DL			
	=(Top container (with water) + shaft + stair case + raft + earth inside and outside)		8475.47	kN
	0.9 DL	=0.9 x 8475.47	7627.93	kN

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	Restoring moment = RM =DL x (raft dia)/2 =8475.47 x 8.4/2	35596.99 kN-m
	Check for safety against overturning	
	Factor of Safety =OM/RM = 4289.62/35596.99 =	8.30
	>1.5 safe Ok	
	Check for safety against sliding	
	Factor of Safety =(0.9DL x μ)/(Max horizontal force) =7628x0.4/182	16.74
	>1.25 safe Ok	

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Wind Load Calculation:

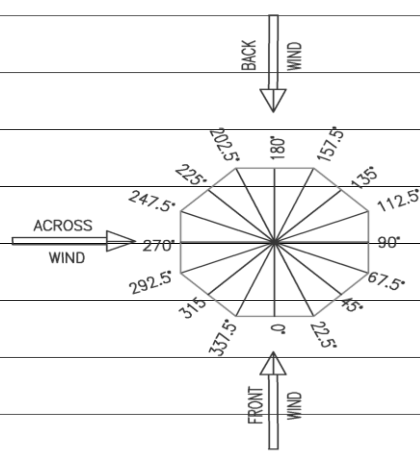
Basic Wind Speed V_b (m/s) =	50	m/s
Risk Coefficient K_1 =	1.08	
Terrain Factor K_2 (For Category-1 & Class-B) =	1.11	
Topography factor K_3 =	1	
Design Wind Speed V_z = $V_b \times K_1 \times K_2 \times K_3$ =	59.94	m/s
Design Wind Pressure acting $P_z = 0.6 \times V_z^2$ =	2155.68	N/m^2
	2.16	kN/m^2

External Pressure Coefficient on shaft and top Cylindrical wall of bowl:

Refer Table-18 (IS: 875 (Part-3) - 1987)

Height of the Tank above ground level (h) =	31.55	m
Outer Diameter of the shaft (D) =	5.25	m
Ratio h/D = 31.55/5.25 =	6.01	

From Table-18 use the coefficients for the nearest curve of h/D =7



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θ in degrees	Shaft (C_{pe})	Wall (C_{pe})
0	1	1
15	0.8	0.8
30	0.1	0.1
45	-0.8	-0.8
60	-1.7	-1.7
75	-2.2	-2.2
90	-2.2	-2.2
105	-1.7	-1.7
120	-0.8	-0.8
135	-0.6	-0.6
150	-0.5	-0.5
165	-0.5	-0.5
180	-0.5	-0.5
195	-0.5	-0.5
210	-0.5	-0.5
225	-0.6	-0.6
240	-0.8	-0.8
255	-1.7	-1.7
270	-2.2	-2.2
285	-2.2	-2.2
300	-1.7	-1.7
315	-0.8	-0.8
330	0.1	0.1
345	0.8	0.8

Internal Pressure Coefficient :

Refer Clause 6.2.3.1 (IS: 875 (Part-3) - 1987)

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	Internal Pressure coefficients for openings not more than 5% (C_{pi}) =				+0.2	
					-0.2	
	Wind Load acting on the shaft (Case-1)					
	θ in degrees	Shaft (C_{pe})	Shaft (C_{pi})	wind force /m l	$F_{along\ wind}$	$F_{across\ wind}$
	0	1	0.2	1.19	1.19	0
	15	0.8	0.2	0.89	0.86	0.23
	30	0.1	0.2	-0.15	-0.13	-0.075
	45	-0.8	0.2	-1.48	-1.047	-1.047
	60	-1.7	0.2	-2.82	-1.41	-2.442
	75	-2.2	0.2	-3.56	-0.921	-3.439
	90	-2.2	0.2	-3.56	0	-3.56
	105	-1.7	0.2	-2.82	0.73	-2.724
	120	-0.8	0.2	-1.48	0.74	-1.282
	135	-0.6	0.2	-1.19	0.841	-0.841
	150	-0.5	0.2	-1.04	0.901	-0.52
	165	-0.5	0.2	-1.04	1.005	-0.269
	180	-0.5	0.2	-1.04	1.04	0
	195	-0.5	0.2	-1.04	1.005	0.269
	210	-0.5	0.2	-1.04	0.901	0.52
	225	-0.6	0.2	-1.19	0.841	0.841
	240	-0.8	0.2	-1.48	0.74	1.282
	255	-1.7	0.2	-2.82	0.73	2.724
	270	-2.2	0.2	-3.56	0	3.56
	285	-2.2	0.2	-3.56	-0.921	3.439
	300	-1.7	0.2	-2.82	-1.41	2.442
	315	-0.8	0.2	-1.48	-1.047	1.047
	330	0.1	0.2	-0.15	-0.13	0.075
	345	0.8	0.2	0.89	0.86	-0.23

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			SUM =	5.37	0

Wind Load acting on the shaft (Case-2)

θ in degrees	Shaft (C_{pe})	Shaft (C_{pi})	wind force /m h	$F_{along\ wind}$	$F_{across\ wind}$
0	1	-0.2	1.78	1.78	0
15	0.8	-0.2	1.48	1.43	0.383
30	0.1	-0.2	0.45	0.39	0.225
45	-0.8	-0.2	-0.89	-0.629	-0.629
60	-1.7	-0.2	-2.23	-1.115	-1.931
75	-2.2	-0.2	-2.97	-0.769	-2.869
90	-2.2	-0.2	-2.97	0	-2.97
105	-1.7	-0.2	-2.23	0.577	-2.154
120	-0.8	-0.2	-0.89	0.445	-0.771
135	-0.6	-0.2	-0.59	0.417	-0.417
150	-0.5	-0.2	-0.45	0.39	-0.225
165	-0.5	-0.2	-0.45	0.435	-0.116
180	-0.5	-0.2	-0.45	0.45	0
195	-0.5	-0.2	-0.45	0.435	0.116
210	-0.5	-0.2	-0.45	0.39	0.225
225	-0.6	-0.2	-0.59	0.417	0.417
240	-0.8	-0.2	-0.89	0.445	0.771
255	-1.7	-0.2	-2.23	0.577	2.154
270	-2.2	-0.2	-2.97	0	2.97
285	-2.2	-0.2	-2.97	-0.769	2.869
300	-1.7	-0.2	-2.23	-1.115	1.931
315	-0.8	-0.2	-0.89	-0.629	0.629
330	0.1	-0.2	0.45	0.39	-0.225
345	0.8	-0.2	1.48	1.43	-0.383
			Σ	5.37	0

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	Total horizontal force acting on the shaft due to wind :			
	Horizontal Wind load acting on the shaft per m height =		5.37	kN/m
	Height of Shaft above the GL upto bottom of Top Bowl =		30	m
	Depth of Shaft below the GL upto top of footing =		2.50	m
	Total Horizontal wind load acting on the shaft =	5.37x30 =	161.1	kN
	Maximum moment acting at the base of the shaft =			
	= 161.1x(30/2+2.5) =		**2819.25**	kN-m
	Force acting on the roof of the structure:			
Ref. Table 19	Structure according to the shape is (c)			
	H=		34.1	
	D=		5	
IS: 875	H/D =		6.82	
(Part 3)-1987	C_{pe} for above value of H/D =		-1	
	C_{pi} =		0	
	Force acting on the roof of the structure, $P = 0.785\ D^2\ (p_i - C_{pe}\ p_d)$ =		42.39	kN
	e = 0.1 D =		0.5	
	Moment about the base of tank =		**21.20**	kN-m
	Total Moment =		**2840.45**	kN-m

CALCULATION OF STRESSES IN SHAFT SECTION AT BASE OF SHAFT
(As per Clause 8.2.5.2 of IS:11682-1985)
Tank Operating condition+SL - Table-1

LEVEL	Width of opening (m)	Grade of concrete	ID m	thk m	Axial load (KN)	Moment KN-m	BETA β (Deg)	ALPHA α (Rad)	BETA (Rad)	Modular ratio (m)
0.000	**1.000**	**30**	4.75	0.250	4240.4	4289.62	11.31	2.583087	0.1973954	9.3300

p	ALPHA (assumed) (Deg)	e m	e/r	A	B	A/B	σ_{cv}' N/mm^2	σ_{cv} N/mm^2	σ_{sy} N/mm^2
0.0025	148	1.01162	0.4	0.811713246	2.033669198	0.4	**2.580**	**2.651** **< 12 ok**	**2.000** **< 249 ok**

(e/r - A/B)= 0.00000

mp	1-p+mp	1-p	sin α cos α	sin β cos β	sin β cos α	sin α	α cos α	sin β	β cos α	mp.π.cos α
0.023325	1.020825	0.9975	-0.449397023	0.192307538	-0.166315776	0.529919	-2.19058	0.196116	-0.1674008	-0.06214292

A	B	B'
0.811713246	2.033669	4.809512

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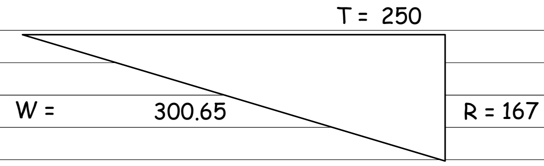
DESIGN OF STAIR CASE

DESIGN OF STAIR CASE

* Maximum span of flight is designed and the same reinforcement is provided for all flights and landing slab.

Design data :

fck	=	25 N/mm²
fy	=	500 N/mm²
Tread , T	=	250 mm
Rise , R	=	167 mm
Thickness of Waist slab , D	=	150 mm



Dead load :

On landing area,	Self wt.of slab	=	3.75 KN/m²
	Finish load	=	1.2 KN/m²
	Total dead load	=	4.95 KN/m²

On Stair area,

Flight load = 1/ T (D * W + T * R / 2) * 25

= 1 / 0.25 (0.15 * 0.30 + 0.25 * 0.17 / 2) * 25

= 6.60 KN/mm²

Span for stair area 2.5 m

Span for landing area =

l_1 = 1 m

l_2 = 1 m

Clause 33.1., IS : 456,

Effective span,

ES = A + B + C = 2.5 m

Live load :

Live on landing & stair area = 4 KN/m²

Factored loads,

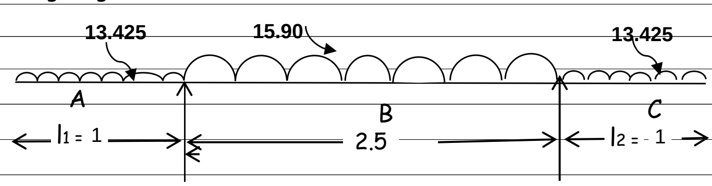
On landing area, = 1.5 * (DL + LL)

= 13.43 KN/m²

On stair area, = 1.5 * (DL + LL)

= 15.90 KN/m²

Loading diagram ,



From staad

	Ra	=	33.33 KN
From staad	Rb	=	33.33 KN

Maximum B.M.

Mu = 7.00 KN-m

	Clear cover in mm =	30 mm
	Assuming dia of bar as =	10 mm
	Effective depth, d =	115 mm
	Table , SP : 16	
	Reinforcement :	
	Mu/bd^2 =	0.53 N/mm^2
	pt =	0.12 %
	Ast(req) =	143.51 mm^2
	Required 10 Dia. @	547 mm c/c
	Provide 10 Dia. @	125 mm c/c
	therefore,	
	pt(prov) =	0.55 %
	Ast(prov) =	628.3 mm^2
	Minimum reinforcement required =(0.12/100)*1000*150	187.2 mm^2
	Provide 8 mm dia 200 mm spacing c/c	251.2 mm^2
	Reinforcement provided	
	pt(prov) =	0.17 %
	Check for shear :	
	Actual shear stress,Vu =	33.33 KN
	T_V =	0.29 N/mm^2
	for pt =	0.55
	Allowable shear stress,T_c =	0.507 N/mm^2
		> T_V
	NO SHEAR REINFORCEMENT IS REQUIRED	
	Check for deflection :	
	(From IS:456:2000 clause 23.2)	
	Allowable span /depth ratio =	20.00
	% of tension reinforcement =	0.55
	fs = 0.58*415*(143.51/628.32) =	54.98
	From Fig 4 Modification factor for tension Rft (Mft) =	2.00
	From Fig 5 Modification factor for tension Rft (Mfc) =	1.00
	Modified span /depth ratio = l/d x Mft x Mfc =	40.00
	Actual span/depth ratio 2.5*1000/115 =	21.74
	Actual span/depth ratio < Modified span/depth ratio =	safe